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Benchmarking of 2D Hydraulic Modelling Packages

SC080035/SR2

Flood and Coastal Erosion Risk Management Research and Development Programme

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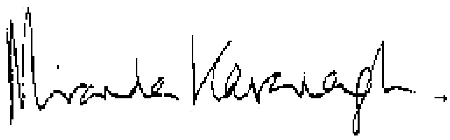

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Director of Evidence

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Executive summary

Environment Agency Science Report, SC080035/SR, “Desktop review of 2D hydraulic modelling packages”, (© copyright: Environment Agency, 2009) discusses the theoretical background to 2D flood inundation modelling and makes recommendations for benchmark test cases to differentiate between 2D model types in terms of performance and predictive capability. The work reported here builds on this through further development of the benchmark test cases, the application of a range of software packages to these tests and comparative reporting of the outcome from the tests. To differentiate between packages this report makes reference to the theories and methods used in 2D flood inundation modelling, those readers requiring an explanation of the theoretical aspects of flood modelling are referred to SC080035/SR.

The objectives of this project are to provide:

1. Evidence to ensure that 2D hydraulic modelling packages used for flood risk management, by the Environment Agency and their consultants, are capable of adequately predicting the variables upon which flood risk management decisions are based.
2. A data set against which such packages can be evaluated by their developers in the future.

An open invitation to participate in the exercise was issued to all developers of 2D flood inundation software known to be applied in the UK. This resulted in a positive response from the suppliers of fourteen software packages. Of these ANUGA, FloodFlow, Infoworks 2D, ISIS2D, MIKE FLOOD, SOBEK, TUFLOW and TUFLOW FV solve the shallow water equations. The shallow water equations include a mathematical description of all the physical processes thought to control the movement of flood waves in two spatial dimensions. In the report software packages of this type have been referred to as using the “full” equations. There are packages where some of these terms are neglected and the software solves simplified equations. The rationale for this approach is that solving simpler equations requires less computer resource and the simulations will therefore require less run time. Packages in this category that participated in the exercise are:

1. JFLOW-GPU, and UIM which solve the 2D diffusion wave equation, obtained by neglecting the acceleration terms in the 2D shallow water equations.

2. RFSM (Direct), employs a technique based on continuity and topographic connectivity. It predicts the ‘final’ state of inundation only (i.e. no variations in time)

3. Flood Risk Mapper, Flowroute and RFSM (Dynamic), use continuity to distribute flood volume between storage areas and then compute the flow rates between these using Manning’s equation (or alternatively a weir flow equation in the case of the RFSM). Flood Risk Mapper and Flowroute rely on a square grid whereas RFSM can also be applied on an irregular grid (user-defined or automatically generated).

It is important to note that the version of JFLOW used in the benchmarking exercise is JFLOW 7.1 GPU. This is a significantly different package from JFLOW 7.0 CPU currently used internally within the Environment Agency. JFLOW was developed specifically for the Environment Agency 2004 Extreme Flood Outline study and JFLOW 7.0 CPU (and its upgrade JFLOW 7.1 CPU) was provided for internal Environment Agency use to support this activity. Environment Agency users of JFLOW 7.0 CPU and JFLOW 7.1 CPU should not draw any conclusions on their fitness for purpose from the JFLOW 7.1 GPU simulations reported here.

The main conclusions from the study are that, packages based on the shallow water equations are appropriate to support decision making across the full range of Environment Agency flood risk management activities making. Exceptions to this apply where:

1) The area of application is large (e.g. 1000 Km^2) or a probabilistic approach requiring multiple simulations is required. In such instances, the time taken to run shallow water equation simulations may be prohibitively long;

2) Where the detail of supercritical to subcritical flow transition is required, such as, in areas close to a dam or embankment breach. If this level of detail is required the numerical scheme used by the software has an influence on capturing the detail of the flow field. The results indicate that packages which employ a shock capturing numerical scheme (ANUGA, InfoWorks 2D, ISIS2D [TVD version], SOBEK and TUFLOW FV) perform better overall in such circumstances.

Water levels predicted by packages based on simplified equations, Flowroute, JFLOW-GPU and UIM, are comparable to those predicted by shallow water equation packages. Where their performance is less comparable is in the prediction of velocities (with predictions often oscillating in rapidly varying flows), and in areas where momentum conservation is important, such as the prediction of water levels and velocities in the complex flow field close to a dam failure and where the spreading flood encounters an adverse slope on the floodplain. The comparisons of run-times indicate that there is no consistent saving in computational effort in applying simplified equation packages compared to shallow water equation packages for the tests reported here. However, this may be a consequence of the scale of the tests used here which are over smaller domains than one would typically apply a simplified model to.

Where clear flow paths across the floodplain exist, RFSM (Direct) produces predictions of final inundation extent and depth that compare well with shallow water equation packages. For more complex topographies the comparisons diverge. This limits the application of RFSM to relatively large scale applications where dynamic effects are less significant in determining the direction of water movement. In all cases RFSM (Direct) requires significantly less computer effort than the other software packages.

FloodFlow predictions deviated from the benchmark test specifications through the use of a depth varying value of hydraulic roughness. As a result the predictions are not directly comparable with those from the other packages and it was not possible to draw quantitative conclusions of its performance relative to the other packages.

The water level predictions made by Flood Risk Mapper and RFSM (Dynamic) show considerable variation with those from shallow water equation models for water level predictions (except for tests 2 and 4). Further work on these packages is necessary before they can be reliably applied to Environment Agency problems.

The benchmark comparisons also highlight a number of other issues of practical relevance to Environment Agency flood risk modelling.

Firstly, where 1D to 2D model linking is used to simulate river to floodplain flood volume exchange, the packages applied used different methods to simulate the hydraulic connectivity between the river and the floodplain. This resulted in significantly different predictions of the volume of water exchanged between the river and the floodplain. This has a knock on effect to the prediction of floodplain inundation and velocity. Predictions made using 1D river to 2D floodplain linking are therefore unlikely to be consistent between software packages. Further research is required to better understand the significance of this.

Secondly, significant differences (up to 100%) in velocity predictions were obtained for high resolution (2m grid) inundation modelling in urban areas. This suggests that a 2m grid is insufficiently fine to adequately resolve the underlying topography for this class of simulation and that, predictions of velocity will not be consistent between packages when applied to the same problem at grid resolutions greater than 2m.

Finally, when applied to simulate large scale valley inundation following a dam break predictions from TUFLOW, ANUGA, and to a lesser extent MIKE FLOOD oscillate in some locations. This results in higher water level and velocity predictions than those obtained by the other shallow water equation packages. Using these predictions to create maps of maximum inundation and velocity could result in exaggerated inundation extent and velocity magnitudes.

For further information on accessing the benchmarking data and the results from the study please contact: fcerm.evidence@environment-agency.gov.uk.

The results and conclusions in this report are accurate at the time of publication, but they represent a ‘snap-shot’ in time. It is likely that development work will be undertaken on the software packages discussed in this report, and so in time it is possible that the results and conclusions may become less relevant to individual software packages. However, the conclusions which compare the generic use of models using the full equations and those using the simplified equations will probably be relevant over a longer time period.

1. Introduction

Environment Agency Science Report, SC080035/SR, “Desktop review of 2D hydraulic modelling packages”, (© copyright: Environment Agency, 2009) discusses the theoretical background to 2D flood inundation modelling and makes recommendations for benchmark test cases to differentiate between 2D model types in terms of performance and predictive capability. The work reported here builds on this through further development of the benchmark test cases, the application of a range of software packages to these tests and comparative reporting of the outcome from the tests. To differentiate between packages this report makes reference to the theories and methods used in 2D flood inundation modelling, those readers requiring an explanation of theoretical aspects of flood modelling are referred to SC080035/SR.

The objectives of this project are to provide:

1. Evidence to ensure that 2D hydraulic modelling packages used for flood risk management, by the Environment Agency and their consultants, are capable of adequately predicting the variables upon which flood risk management decisions are based.
2. A data set against which such packages can be evaluated by their developers.

A number of software packages for 2D inundation modelling are available and the performance and predictive capability of each is controlled by both, the suitability of the software to undertake the modelling task, and the skill of the modeller in building the model. In turn suitability of the software depends upon:

1. the mathematical formulation of the physical processes controlling flood movement across a floodplain;
2. the numerical method used to solve the mathematical formulation; and,
3. the configuration of the numerical grid upon which the numerical solution is applied.

Whereas, modeller skill encapsulates the personal judgement of the modeller to ensure:

1. appropriate representation of boundary conditions (inflows to and outflows from) the modelled domain;
2. correct representation of problem geometry on the numerical grid upon which the numerical method is applied;
3. model calibration and choice of model parameters, such as boundary roughness and choice of time increment.

There is a significant body of scientific literature discussing many of the above issues but little of it evaluates how the underlying assumptions made in software development impact on practical decision making at the level required by the Environment Agency. This report attempts to evaluate the importance of this issue through a quantitative evaluation of a wide range of software developed for 2D flood inundation prediction. To achieve this, a range of 2D flood inundation software packages have been applied to eight closely specified benchmark test cases, summarised in

Table 1 and described in detail in Appendix A. As the purpose of the exercise is to evaluate software performance rather than the inherent ability of individual modellers, each test has been tightly specified to limit the need for modeller skill in making model predictions. The exceptions to this approach are tests 7 and 8 where the practical nature of the test requires a limited amount of judgement to be applied in creating the models. A discussion of the judgements made for each package is present in Appendix C.

2. Benchmarking tests

There are a range of possible approaches to benchmarking including comparing hydraulic model predictions with analytical solutions, field data, physical model data and other model predictions of real or hypothetical flood events. Report SC080035/SR “Desktop review of 2D hydraulic modelling packages” discusses the advantages and disadvantages of each of these approaches. In consultation with the software developers the outline test cases presented in SC080035/SR were amended to the eight tests reported here. These are summarised in

Table 1 and described in detail in Appendix A.

Table 1: Summary of benchmark tests

Number	Description	Purpose
1	Flooding a disconnected water body	Assess basic capability to simulate flooding of disconnected water bodies on floodplains or coastal areas.
2	Filling of floodplain depressions	Tests capability to predict inundation extent and final flood depth for low momentum flow over complex topographies.
3	Momentum conservation over a small (0.25m) obstruction.	Tests capability to simulate flow at relatively low depths over an obstruction with an adverse slope.
4	Speed of flood propagation over an extended floodplain.	Tests simulation of speed of propagation of flood wave and the prediction of velocities at the leading edge of the advancing flood.
5	Valley flooding	Tests simulation of major flood inundation at the valley scale.
6A & 6B	Dam break	Tests simulation of shocks and wake zones close to a failing dam.
7	River to floodplain linking	Evaluates capability to simulate flood volume transfer between rivers and floodplains using 1D to 2D model linking.
8A & 8B	Rainfall and sewer surcharge flood in urban areas	Tests capability to simulate shallow flows in urban areas with inputs from rainfall (8A) and sewer surcharge (8B).

As shown in Table 2 these have been designed and specified to evaluate software suitability for Environment Agency needs.

Table 2: Mapping of benchmark test case to model type and Environment Agency application

Appl No.	Application	Predictions required	Relevant benchmark test
1	Large[1] Scale Flood Risk Mapping	i. inundation extent	1 & 2
2	Catchment Flood Management Plan	ii. inundation extent iii. maximum depth	1, 2 & 7
3	Flood Risk Assessment and detailed flood mapping	i. inundation extent ii. maximum depth	1, 2, 3 and 7
4	Strategic Flood Risk Assessment	i. inundation extent ii. maximum depth iii. maximum velocity	1, 2, 3, 4 7, and 8.
5	Flood Hazard Mapping	i. inundation extent ii. maximum depth iii. maximum velocity	1, 2 3, 4, 7 and 8
6	Contingency Planning for Real Time Flood Risk Management	i. temporal variation in inundation extent ii. temporal variation in depth iii. temporal variation in velocity	1, 2 3, 4, 5, 7 and 8
7	Reservoir Inundation Mapping	i. temporal variation in inundation extent ii. temporal variation in depth iii. temporal variation in velocity	1, 2, 3, 4, 5, and 6.

1 Large scale can extend to catchments of 1000s km^2

3. Participating software packages

The questionnaire survey outcome presented in Science Report SC080035/SR “Desktop review of 2D hydraulic modelling packages” identified TUFLOW, Infoworks2D, MIKE21 and JFLOW as the 2D packages most commonly applied to Environment Agency problems. This report therefore recommended that these packages be benchmarked against eight standard test cases. At the outset of the current project however, an open invitation was issued to all developers of 2D flood inundation software applied in the UK. The invitation was to participate in the exercise at the developers own cost. This resulted in a positive response from the suppliers of the fourteen software packages listed in Table 3.

The authors and the Environment Agency would like to express their gratitude to the following people and organisations for their contribution in generating the model predictions reported here: Bill Syme, WBM; David Fortune, MWHSoft Ltd, Tyrone Parkinson, MWHSoft Ltd; Ruth Clarke, MWHSoft Ltd; Jonatan Mulet-Marti, MWHSoft Ltd; Paul Sayers HR Wallingford Ltd; David Martin, Ambiental; Aidan Millerick, Micro Drainage; David Wells, Micro-drainage; Slobodan Djordjevic, University of Exeter; Albert Chen, University of Exeter; Morten Rungø, Danish Hydraulic Institute; Steve Flood, Danish Hydraulics Institute; Matt Horritt, Halcrow Group Ltd; Peter Wells, Halcrow Group Ltd; Neil Hunter, JBA Consulting Ltd; Julien Lhomme, HR Wallingford Ltd; Thieu Van Mierlo, Deltares; Edward Melger, Deltares; Miriam Middelman, Geoscience Australia and Nathan Muggeridge, Mouchel.

At the present time the shallow water equations are considered to provide a “full” mathematical representation of the physical processes controlling floodplain inundation at the scale of interest to the Environment Agency. Of the software packages reported in this study ANUGA, FloodFlow, Infoworks 2D, ISIS2D, MIKE FLOOD, SOBEK, TUFLOW and TUFLOW FV solve the shallow water equations, including the acceleration, pressure, bottom slope, and friction slope terms. There are instances however, where software solving simpler equations may be appropriate. Packages in this category that are reported here are:

1. JFLOW-GPU, and UIM which solve the 2D diffusion wave equation which is obtained by neglecting the acceleration terms in the 2D shallow water equations.

2. RFSM (Direct): a volume spreading algorithm which effectively only considers continuity and topographic connectivity. It predicts only the ‘final’ state of inundation (i.e. no variations in time)

3. Flood Risk Mapper, Flowroute and RFSM (Dynamic): use continuity to distribute flood volume between storage areas and compute the flow rates using Manning’s equation (or alternatively a weir flow equation in the case of the RFSM). Flood Risk Mapper and Flowroute are applied on a square grid whereas RFSM can also be applied on an irregular grid (user-defined or automatically generated).

It is important to note that the version of JFLOW used in the benchmarking exercise is JFLOW 7.1 GPU. This is a significantly different software package from JFLOW 7.0 CPU currently used internally within the Environment Agency. JFLOW was developed specifically for the 2004 Extreme Flood Outline study and JFLOW 7.0 CPU (and its upgrade JFLOW 7.1 CPU) was provided for internal Environment Agency use to support this activity. Environment Agency users of JFLOW 7.0 CPU and JFLOW 7.1 CPU should not draw any conclusions on their fitness for purpose from the JFLOW 7.1 GPU simulations report here.

Table 3 contains a brief summary of the technical attributes of each of these packages. The information in columns (3), (4), (5) has been provided mainly by the suppliers. Where this is not the case the data has been extracted from publicly available sources (internet, user’s manuals or scientific publications).

Some packages offer users a choice between two or more numerical methods. This is noted in column (3) where the developers have employed different numerical schemes for different tests, so, for example, the Finite Volume solver available with MIKE FLOOD is not listed as only the Finite Differences solver was used in the test application.

Column (5) reports whether the 2D inundation modelling software can be linked to 1D river and/or pipe flow elements. The designation used for some of the participating packages in fact refers to an integrated 1D/2D package rather than to a 2D solver only.

The information in column (6) (Minimum recommended hardware) was provided by the software developers and is reproduced here without modification.

The following packages in Table 3, FloodFlow, InfoWorks2D, ISIS2D, MIKE FLOOD, SOBEK, TUFLOW and TUFLOW FV are commercially available or in development for future commercial release. Those presently limited to application by their developers are, Flood Risk Mapper, Flowroute, JFLOW-GPU, RFSM and UIM. RFSM has been applied indirectly by Environment Agency staff via the RASP and MDSF tools. ANUGA is an open source application.

Table 3: Summary of software packages considered in the present report.

(1) Name	(2) Developer	(3) Numerical scheme(s)[1]	(4)Shock Capturing	(5) 1D-2D Linkages	(6) Minimum recommended hardware specification
ANUGA	Geoscience Australia & Australian National University[2]	FV	Yes	No	Windows or Linux PC RAM: 512 MB
Flood Risk Mapper	Mouchel	FD (Explicit)	No	No	Not provided by supplier
FloodFlow	Micro Drainage	FD (ADI or Explicit)	No	Yes (River / overland flow link through a *vertical* link).	Processor: 1GHz RAM: 1 GB
Flowroute	Ambiental	FD (Explicit)	No	No	Processor: 1GHz RAM: 2GB
InfoWorks	Wallingford Software	FV (Roe's Riemann solver)	Yes	Yes. Integrated 1D-2D package	Processor: Pentium 4.1.5GHz RAM: 2GB
ISIS	Halcrow	FD (ADI or TVD)	Yes (TVD only)	Yes. Integrated 1D-2D package	Processor: Intel Pentium 4 or equivalent RAM: 100 MB
JFLOW-GPU	JBA	FD (Explicit)	No	No	The GPU should be a NVIDIA G80 series or a subsequent card. Any machine capable of supporting the graphics card.
MIKE FLOOD	DHI	FD (ADI)	No	Yes. Integrated 1D-2D package	Processor type: Intel or AMD Speed: 2.0 GHz RAM: 2.00 GB

[1] As used in project.
[2] ANUGA was introduced to the project by HALCROW and the tests were performed by HALCROW.

(1) Name	(2) Developer	(3) Numerical scheme(s)[1]	(4)Shock Capturing	(5) 1D-2D Linkages	(6) Minimum recommended hardware specification
RFSM (Direct)	HR Wallingford	N/A[3]	No	No	No specific needs - a standard recent computer.
RFSM (Dynamic)	HR Wallingford	FD (Explicit)	No	No	No specific needs - a standard recent computer.
SOBEK	Deltares	FD (Implicit - Staggered grid)	Yes	Yes. Integrated 1D-2D package[4].	Processor type: Pentium or compatible Processor speed: 1 GHz RAM: min 500 MB, recommended 2GB
TUFLOW	BMT WBM	FD (ADI)	No	Yes. Integrated 1D-2D package	Any PC
TUFLOW FV	BMT WBM	FV[5]	Yes	No	Any Windows or Linux PC
UIM	University of Exeter	FD (Explicit)	No	Yes. Integrated with 1D sewer network model (SIPSON)	Desktop PC

[3] As the RFSM Direct does not rely on differential equations.
[4] Allows for both *vertical* and *horizontal* links between 2D overland flow and 1D flow
[5] For flexible meshes comprising quadrilateral and triangular elements.

4. Outcome of the benchmarking exercise

4.0 Introduction

This section contains a summary of the predictions from each package for the eight benchmarking tests discussed in Section 2 and present in detail in Appendix A.

4.0.1 Participation in tests

Not all of the tests were undertaken by all participants, as detailed in Table 4.

Table 4: Summary of participation in benchmarking exercise (indicated by +), with reasons for not undertaking individual test (as provided by participants`) explained below.

	1	2	3	4	5	6A	6B	7	8A	8B
ANUGA	+	+	+	+	+	+	+	(2)or(5)	(2)	(2)or(6)
Flood Risk Mapper	(1)	+	+	+	(2)	(3)	(3)	(5)	+	(6)
FloodFlow	+	+	(1)	+	+	(4)	(3)	+	+	+
Flowroute	+	+	+	+	+	(1)(3)	(1)(3)	(5)	+	(6)
InfoWorks 2D	+	+	+	+	+	+	+	+	+	+
ISIS 2D	+	+	+	+	+	+	+	+	+	+
JFLOW-GPU	(1)	+	+	+	+	(3)(4)	+	(5)	+	(6)
MIKE FLOOD	+	+	+	+	+	+	+	+	+	+
RFSM Direct	(1)	+	+	(3)	+	(1)(3)	(1)(3)	(5)	+	(6)
RFSM Dynamic	+	+	(3)	+	+	(1)(3)	(1)(3)	(5)	+	(6)
SOBEK	+	+	+	+	+	+	+	+	+	+
TUFLOW	+	+	+	+	+	+	+	+	+	+
TUFLOW FV	+	+	+	+	+	+	+	(5)	+	(6)
UIM	+	+	+	+	+	(3)	(3)	(5)	+	+

(1): Type of boundary or initial condition not supported.

(2): Resources were not available to undertake this test in the required time frame.

(3): Model unlikely to produce useful results.

(4): Scale of test too small for software.

(5): Linked 1D River + 2D Floodplain modelling not supported (or in development)

(6): Linked 1D Pipe + 2D Floodplain modelling not supported (or in development)

The conclusions drawn are based on the current capabilities of the packages participating. It should be noted that future releases of the packages may include the ability to run the Tests for which (1), (4), (5) or (6) is given as a reason for not participating. At a later date developers may conduct and report new outcomes for the tests resulting from the ongoing development of their software. No conclusions can be drawn at this stage if (2) is indicated as the reason for non-participation. However, where (3) is indicated as the reason for non-participating, relevant comments are made in the conclusions, mentioning the package's limitations.

For Flood Risk Mapper, RFSM Dynamic, and RFSM no velocity predictions were supplied for the reasons below provided by the developers:

Flood Risk Mapper: "We have identified an issue with how the velocity data is being written by the programme. It is being calculated correctly, but the reporting needs to be changed and this will be done in due course. We have subsequently not provided any velocity information." (Mouchel, 1st February 2010)

RFSM Dynamic: "Velocities have not been provided, they are calculated by the algorithm but we are not currently satisfied by the level of accuracy of those." (HR Wallingford, 1st February 2010)

RFSM Direct: Velocities are not calculated.

4.0.2 Miscellaneous information received from participants

Comments that the participating software developers provided as part of their submission of results are presented in Appendix B. These should be considered when interpreting the test results and cover most tests. Additional comments specific to individual tests were also provided, and have been included in Sections 4.1 to 4.8.

The reader's attention is drawn particularly to the simplified representation of floodplain processes on which Flood Risk Mapper, Flowroute, JFLOW-GPU, RFSM (Direct and Dynamic) and UIM are based.

The specified Manning's n values were not applied in the FloodFlow simulations. Instead a depth-dependent formulation for Manning's n was used, see Appendix B.1, which did not allow a full comparison of the FloodFlow results with others. Therefore FloodFlow results and a discussion of these are presented in Appendix D. Additional FloodFlow results using a constant Manning's n with the specified value were supplied for Test 1 only and are presented below in Section 4.1.

4.0.3 Structure and content of results sections

Model results from Tests 1 to 8B are presented in detail in Sections 4.1 to 4.8. Each of these sections is structured as follows;

- Subsection 1: Brief introduction reminding the reader of the main features of the test, its purpose and expected outcomes.
- Subsection 2: Comparative plots containing all the results provided in the form of time series, followed by a summary of observations arising from the plots.
- Subsection 3: for the tests where results in raster format where required, comparative maps presenting some of these results, followed by additional observations arising from these.
- Penultimate subsection: the main conclusions that can be drawn from the tests.
- Final subsection: Information on the model run as required by the test specification (hardware, time-stepping, grid resolution, run times, etc.), and any relevant information specified by the participants specific to the test.

The plots in Subsection 2 illustrate all the time series as supplied by participants with no further processing. Each graph has a legend clarifying the presence or not of a package's prediction on the graph. The absence of a package on a graph is explained by one of the following reasons:

- The test was not run or results were not supplied by the developers (for reasons as summarised in Section 4.0.1)
- The test was run but only water levels were predicted or supplied (Flood Risk Mapper and RFSM predictions)
- The model predicted the ground to remain dry and time series for output were not supplied (although usually for such points participants supplied a time series consisting of a constant elevation at the level of the ground, as shown by the graphs)

Some curves span less than the specified test duration. This is either because the test was stopped before the specified end or because the model predicted the ground to be dry.

In Subsection 2 comments on the graphs are separated (where appropriate) in two paragraphs concerning 1) the shallow water equation “Full models” and 2) the “Simplified models”. Packages refered to as “full” models are ANUGA, InfoWorks 2D, ISIS 2D, MIKE FLOOD, SOBEK, TUFLOW, and TUFLOW FV. The packages referred to as “Simplified models” are those not doing so, i.e. Flood Risk Mapper, Flowroute, JFLOW-GPU, RFSM Direct, RFSM Dynamic, and UIM. Results from FloodFlow are presented separately in Appendix D. Any comments made or conclusions drawn in Section 4 *do not apply to FloodFlow.*

The results in raster format presented in Subsection 3 could not be presented as comprehensively as the time series predictions due to the practicalities of doing so within a limited space. Instead, the data have been processed in a GIS to create comparative plots of cross-sections and comparative contour plots.

The content of the table presented in the final subsection is mostly self-explanatory. In addition to the package version, Column (2) in the table shows the numerical scheme used in the package, only for packages where more than 1 numerical scheme was used in the benchmarking exercise (the reader is otherwise referred to Table 3). Column (4) in the summary table states whether multi-processing (implying parallel coding) was used[6]. This is relevant to the understanding of the run times presented in Column (7). Column (5) contains information on the numerical grid (resolution or number of elements used), with a reminder of the expected resolution in the header row. The mention ‘Not supplied’ in the Table refers to missing information for tests for which simulations were not undertaken. The tests not undertaken are indicated wherever applicable in Column (2) by ‘Not tested’.

Results from TUFLOW FV were provided with two different numerical approaches (1st order and 2nd order discretisation). Only the results using the 2nd order discretisation are presented in the report. However the run times from the 1st order simulations are presented in the summary Tables in Column (7) in brackets, in addition to the run times from the 2nd order simulations[7].

[6] “Partial” in the row concerning ISIS refers to hyper-threading (4 threads) which was used with the ADI solver only. The main hydrodynamic ADI solution algorithm is not parallelised, but some auxiliary processes (pre-processing files for output for example) were able to use multiple threads. The speed improvement is not as significant compared with a full parallel application.

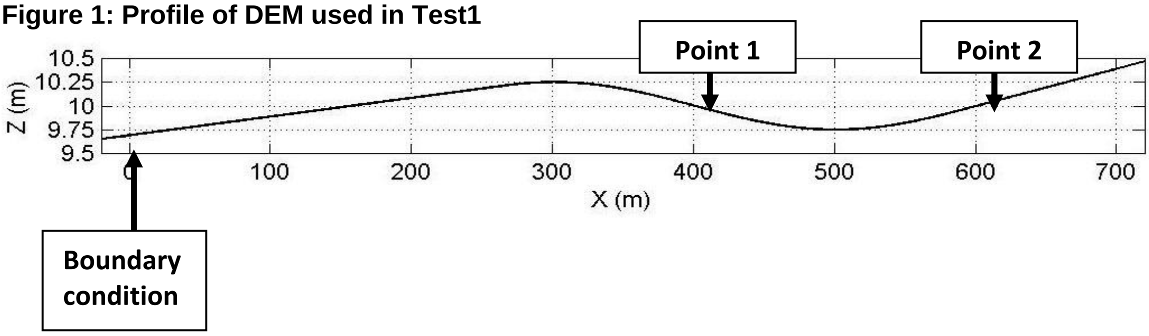
[7] Differences between the 1st order results and the 2nd order results were usually smaller than the differences between the results from the various “full” models presented in the report, except in Test 6.

4.1 Test 1: Flooding a disconnect water body

4.1.1 Introduction

The test (see Appendix A.1 for details) consists of a 100m wide, 700m long domain with a longitudinal profile as illustrated in Figure 1. A water level boundary condition is applied at the left-hand end of the domain, with a peak level of 10.35m maintained for sufficiently long for the depression on the right-hand side to fill up to a level of 10.35m. The level is then lowered to 9.7m at the boundary.

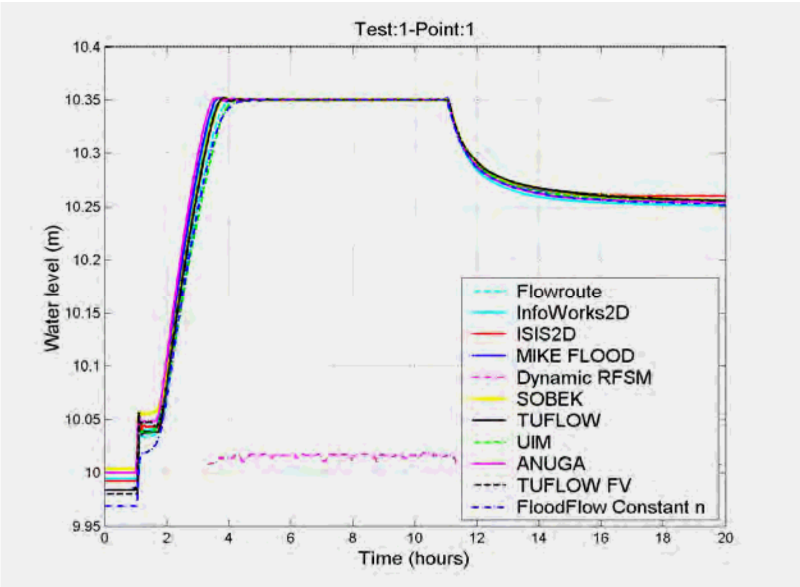
Figure 1: Profile of DEM used in Test1

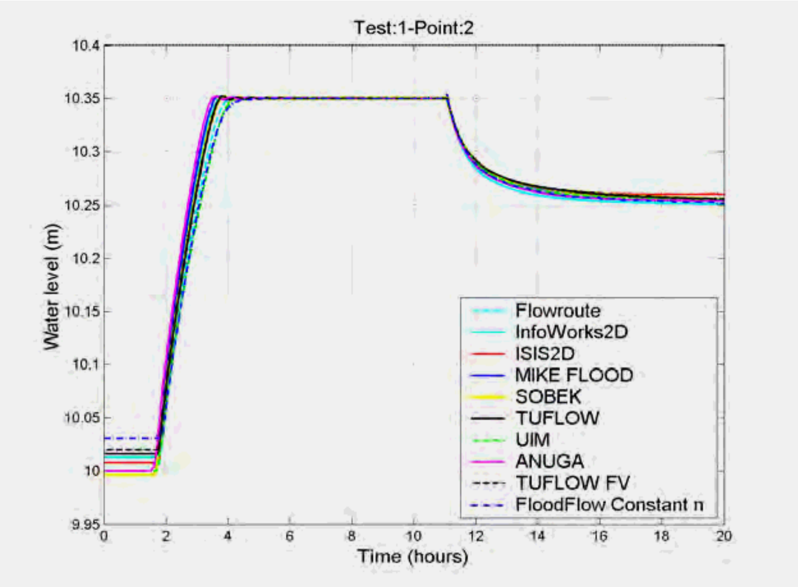


The objective of the test is to assess basic capabilities such as handling disconnected water bodies and wetting and drying of floodplains. Expected outcomes are as follows:

- Peak level of ~10.35m at Points 1 and 2.
- Final level of ~10.25m at Points 1 and 2.

4.1.2 Water level time series





Note: The Dynamic RFSM results are consistent with the prediction of a transient shallow flow at Point 1, caused by water overtopping the obstruction, although not a volume sufficient for the level in the pond to rise above the ground level of Point 1 and 2.

The following observations can be made from the graphs:

- All models predicted an initial sheet flow at Point 1 (depth up to ~5 cm), starting at t≈1hr and lasting for ~45 mins until the water level in the pond reached the elevation of the two output points.
- The water level difference between point 1 and point 2 (located 200m from each other) was negligible (to within a few mm) for all models after t≈2hr.
- The rate of water level rise in the pond, mainly between t≈2hr and t≈4hr was broadly the same between models.
- All models predict a final level elevation of ~10.25m at both points, in accordance with expectations. Discrepancies of up to +0.01m compared to this expected value may be due to the choice of dry/wet threshold depth value.
- An exception to all the above observations is the Dynamic RFSM, which predicted a behaviour very different from any expectations. This is caused by the model's inability to implement the water level boundary condition accurately.

4.1.3 Conclusions from Test 1

All the packages participating in Test 1 (except RFSM Dynamic) demonstrated the basic ability to correctly predict the final state of inundation in a case involving the filling of a depression and subsequent dewatering, resulting in a horizontal water surface in the depression, at the elevation of the lowest point separating the depression from surrounding areas.

No conclusions can be drawn from Test 1 concerning RFSM Dynamic (which failed the test for reasons related to the way the boundary condition is set up in the current version). Flood Risk Mapper, JFLOW-GPU and RFSM Direct did not participate in the test due to their inability to implement the boundary condition.

4.1.4 Summary of relevant technical information

TEST 1 (1) Name	(2) Version (+ numeric. scheme)	(3) Hardware	(4) Multi-Processing	(5) Grid (10m or 700 elem.)	(6) Time-stepping	(7) Run time (s)
ANUGA	1.1beta_7501	Intel Mobile Core 2 Duo T7500 (Merom) 2.2 GHz RAM 2048 MB (DDR2)	no	714 elements	adaptive	205
Flood Risk Mapper	*Not tested*					
FloodFlow	W.12.0 Beta Explicit (D-dependent roughness)	Intel(R) Core™ 2 CPU 6400 2.13GHz RAM 2GB	no	10m	adaptive	300
	As above but constant roughness	As above	no	10m	adaptive	300
Flowroute	2.9.8	2.4Ghz (Intel Q6600) RAM 4GB	OMP	10m	0.05 s	240
InfoWorks	RS 2D v10.5	Intel® Core™ i7 920 2.66GHz Quad Core 12 GB RAM (DDR3-1333)	OMP	714 elements	adaptive	16
ISIS	3.2.0.21 ADI	Intel Core 2-Quad CPU Q6600 2.4 GHz RAM 2.0 GB	Partial, see section 4.0.3	10m	5s	48
JFLOW-GPU	*Not tested*					
MIKE FLOOD	2009 incl. Serv. Pack 2	Intel Core 2 Duo CPU P8600 2.4 GHz RAM 4.00 GB	no	10m	10s	3
RFSM Dir.	*Not tested*					
RFSM (Dynamic)	0.1 (Beta)	Intel Dual Xeon 2 cores of 3GHz RAM 2GB	no	35 elements[8]	30s	72
SOBEK	2.13	Intel i7 8 core CPU 2.66 GHz	no	10m	15s	17
TUFLOW	2010-01-AD-iSP	Intel Core 2 Duo T9800 2.93GHz RAM 4Gb	no	10m	5s	16.6
TUFLOW FV	0.107.0006 2nd order solution	Intel Xeon X5472 3.00GHz RAM 8Gb	Yes – 8 CPUs	10m	Typical time steps: 0.5-0.7sec	25.6 (6.4)
UIM	2009.12	Intel Core™ 2 Duo CPU T7800 2.60GHz RAM 3GB	OMP	10m	0.1s	349

[8] See Appendix B.6.

Other information provided:

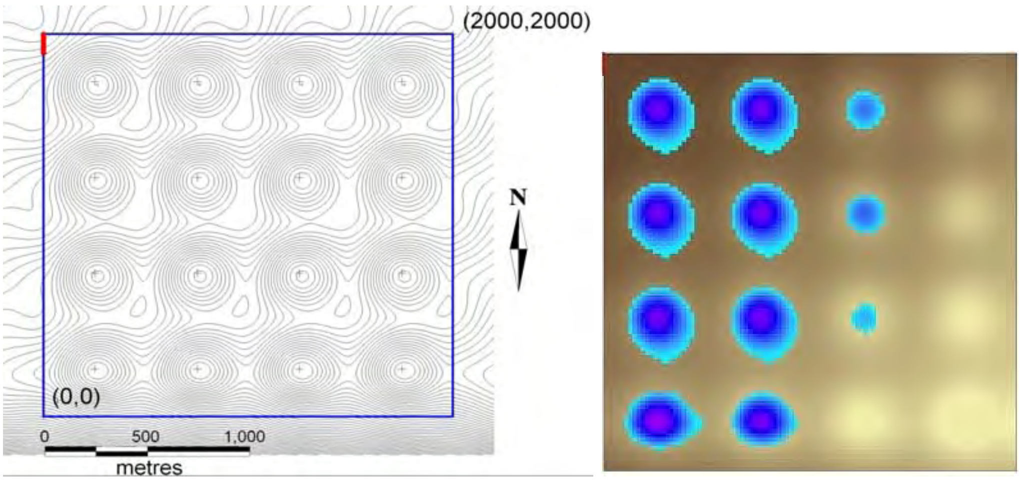
RFSM (Dynamic): “Although a level boundary condition can be defined with the Dynamic RFSM, this feature is not accurate at the present time and is not leading to satisfactory results.”

4.2 Test 2: Filling of floodplain depressions

4.2.1 Introduction

The test (see Appendix A.2 for details) consists of a 2000m × 2000m domain with a 'flattened egg box' shaped topography as illustrated in Figure 2. An inflow hydrograph boundary condition with a peak flow of 20m^3/s and time base of ~85mins is applied at the top left corner of the domain.

Figure 2: Left: map of the DEM showing the upstream boundary condition, ground elevation contour lines every 0.05 m, and output point locations (+'s). Right: final inundation predicted by most models.

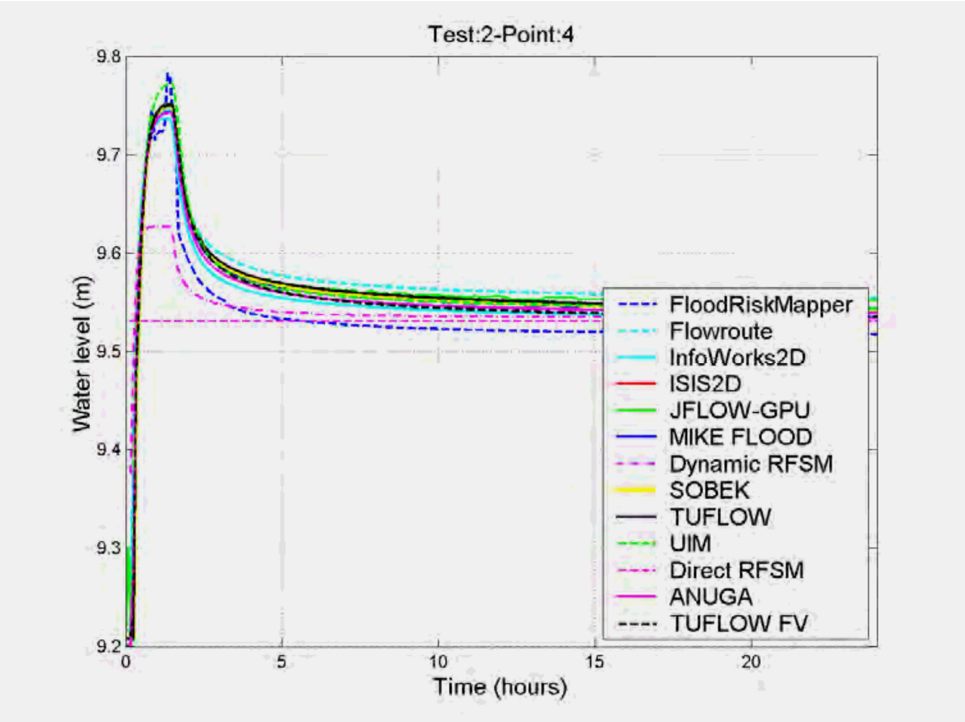


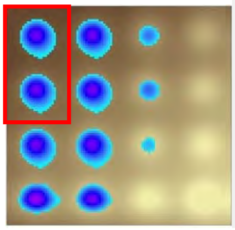
The objective of the test is to assess the package's ability to handle disconnected water bodies, wetting and drying of floodplains, and predict inundation extent due to relatively low momentum flooding on a complex topography, with an emphasis the on final distribution of flood water rather than peak levels.

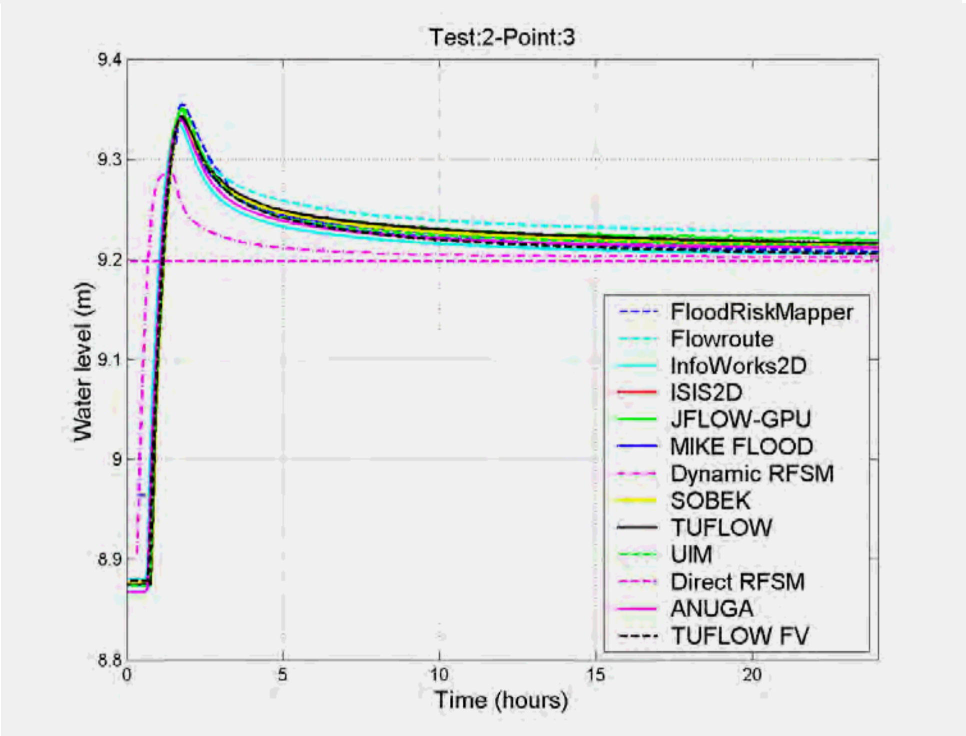
4.2.2 Time series of water levels

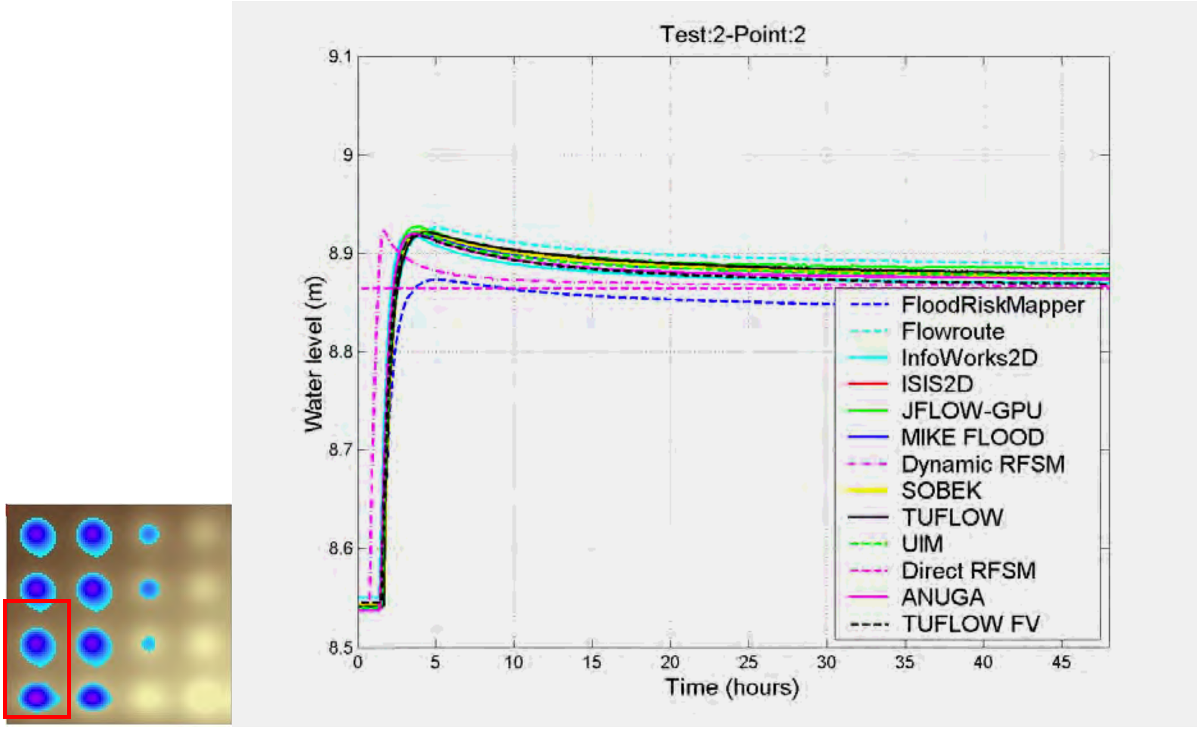
16 output points were specified in Test 2, located at the centres of the 16 depressions. For the purpose of result comparison the depressions are numbered 1 to 16 in columns starting at the bottom left in Figure 2. The following figures represent time series of water levels in the depressions as illustrated on the plan sketch at the top of each page.

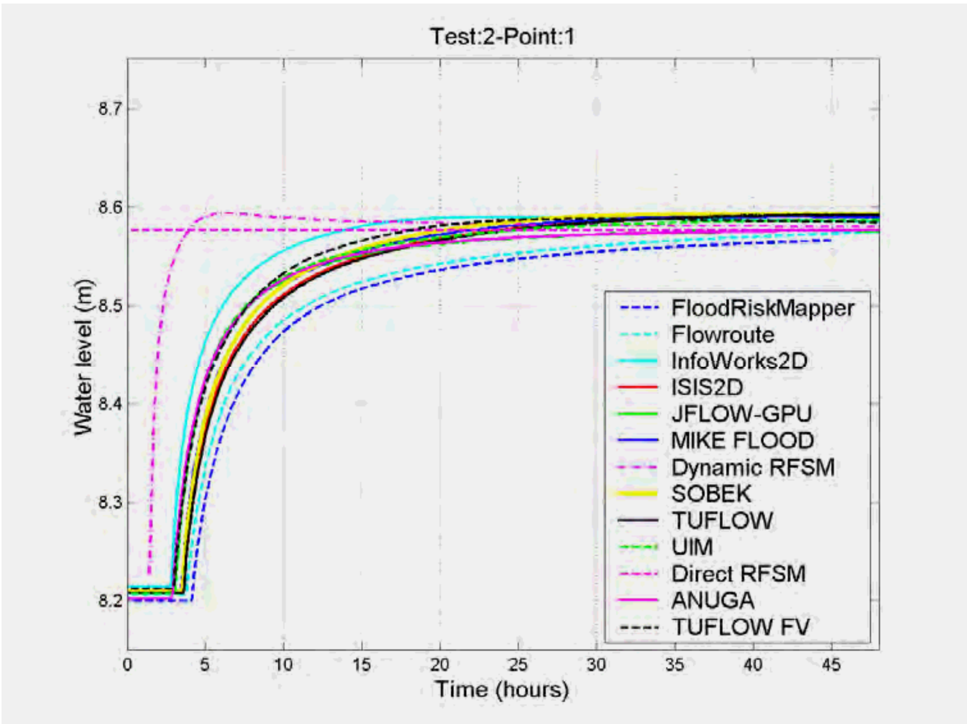
All models predicted the ground to remain dry at points 15 and 16 (top right corner), therefore the time series at these two points are not shown.

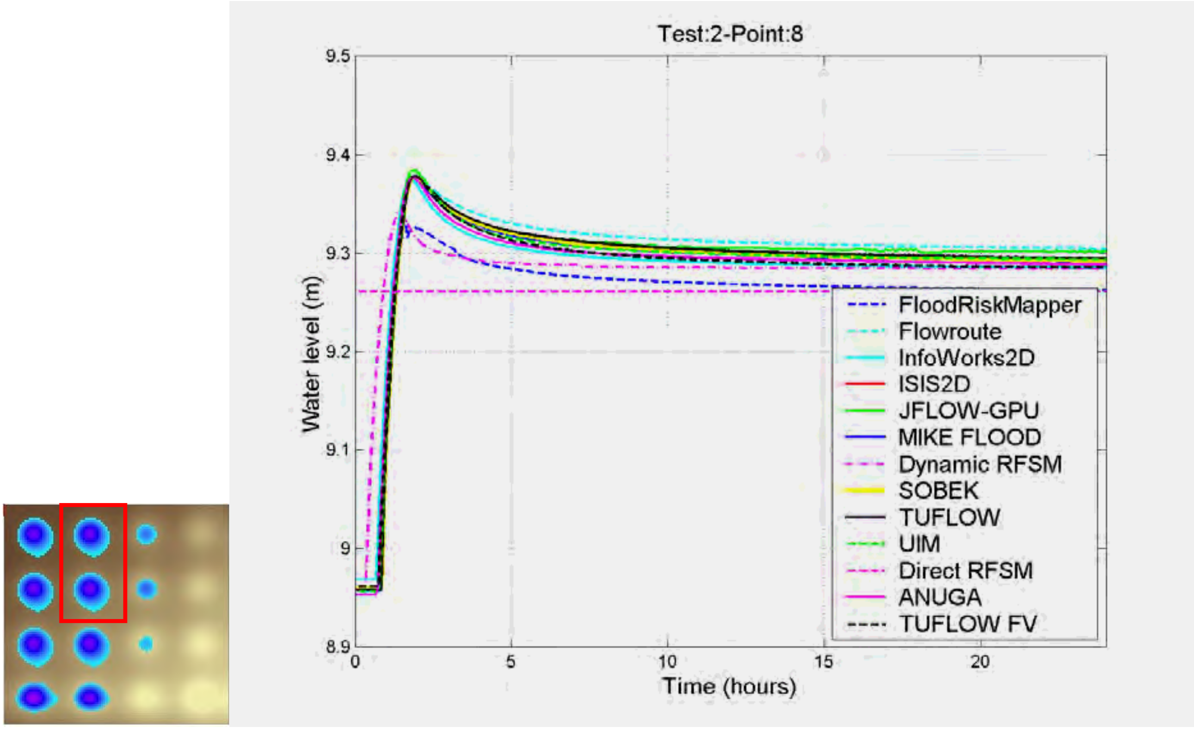

Test:2-Point:4
Water level (m)
Time (hours)
9.8
9.7
9.6
9.5
9.4
9.3
9.2
0
5
10
15
20
FloodRiskMapper
Flowroute
InfoWorks2D
ISIS2D
JFLOW-GPU
MIKE FLOOD
Dynamic RFSM
SOBEK
TUFLOW
UIM
Direct RFSM
ANUGA
TUFLOW FV

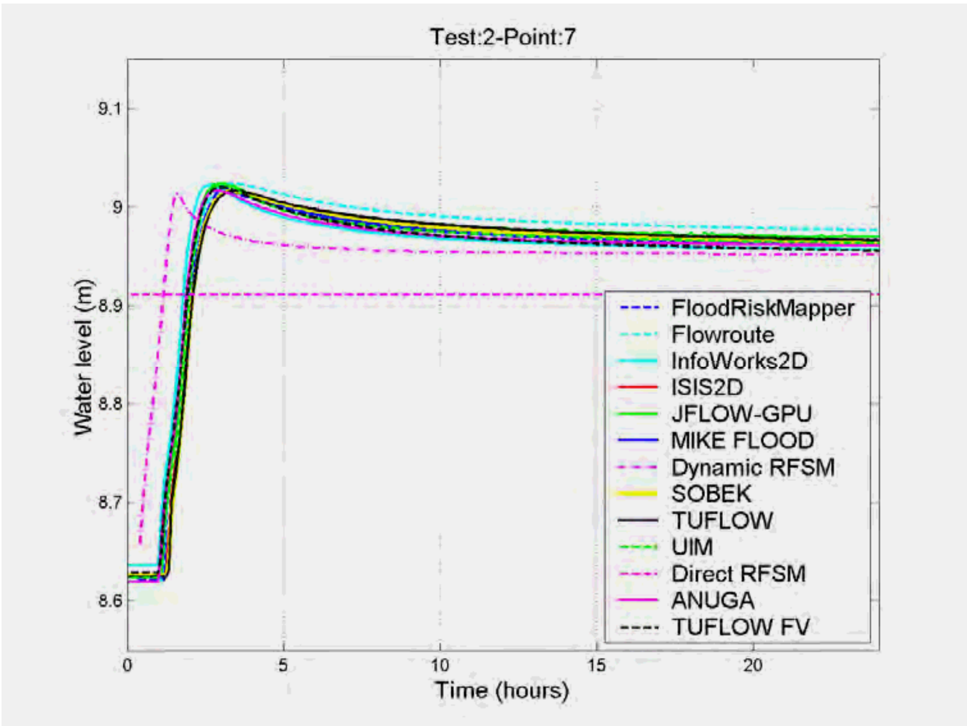


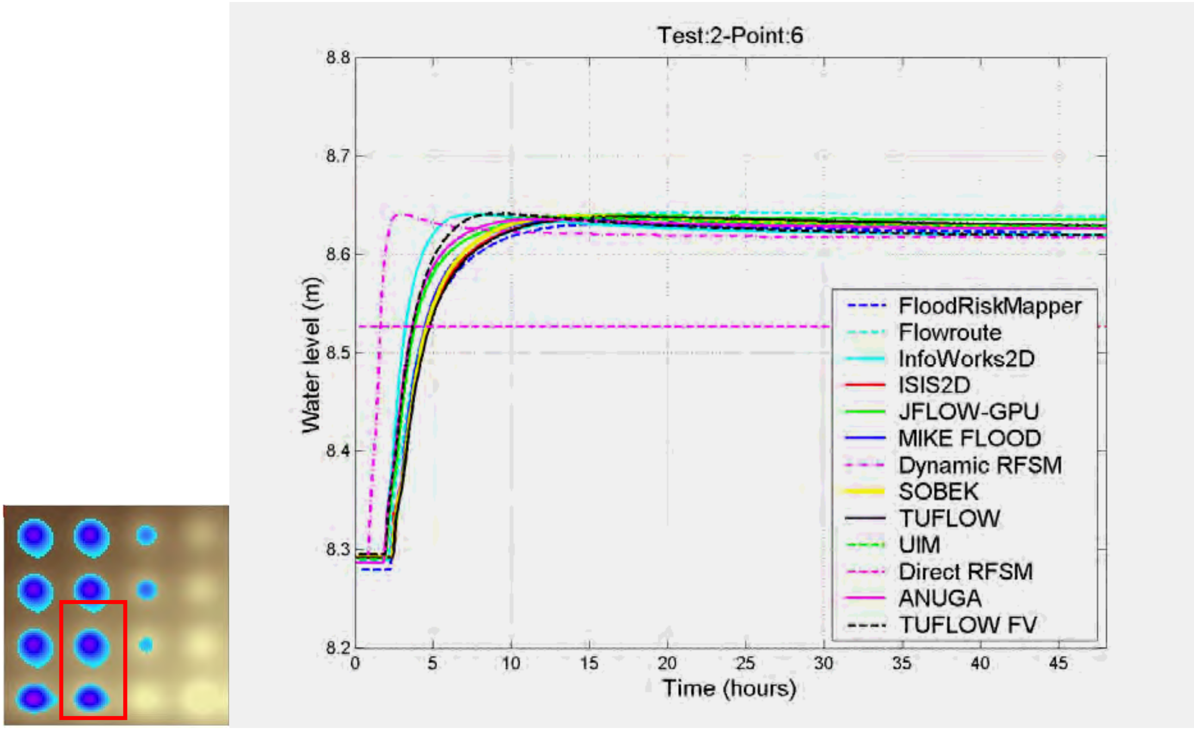

Test:2-Point:3
Water level (m)
Time (hours)
9.4
9.3
9.2
9.1
9
8.9
8.8
0
5
10
15
20
FloodRiskMapper
Flowroute
InfoWorks2D
ISIS2D
JFLOW-GPU
MIKE FLOOD
Dynamic RFSM
SOBEK
TUFLOW
UIM
Direct RFSM
ANUGA
TUFLOW FV

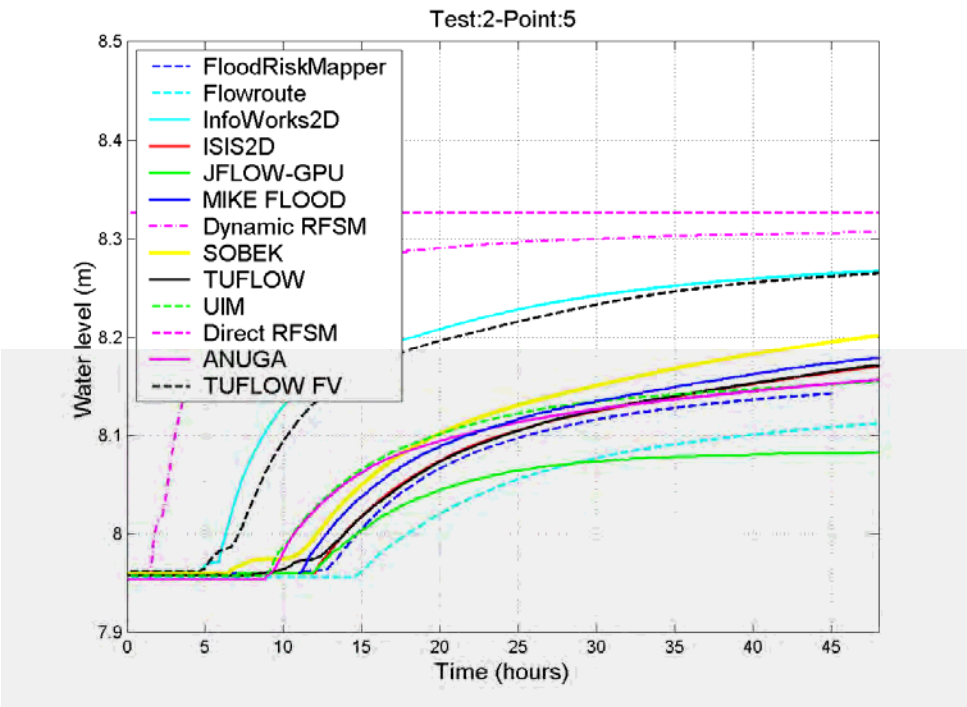

Test:2-Point:2
Water level (m)
Time (hours)
9.1
9
8.9
8.8
8.7
8.6
8.5
0
5
10
15
20
25
30
35
40
45
FloodRiskMapper
Flowroute
InfoWorks2D
ISIS2D
JFLOW-GPU
MIKE FLOOD
Dynamic RFSM
SOBEK
TUFLOW
UIM
Direct RFSM
ANUGA
TUFLOW FV

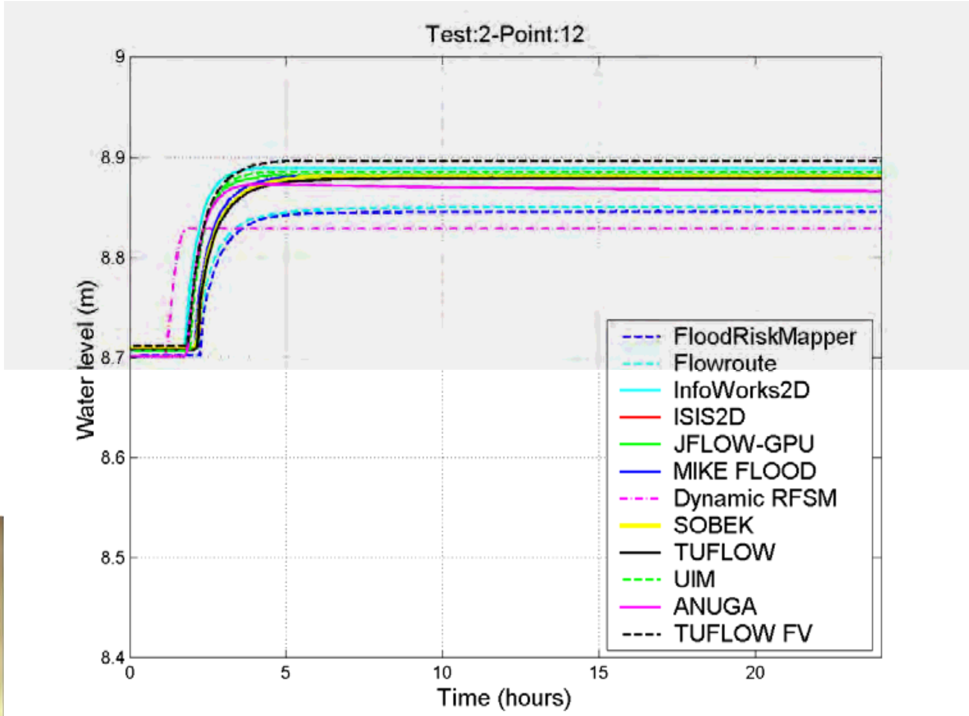

Test:2-Point:1
Water level (m)
Time (hours)
8.7
8.6
8.5
8.4
8.3
8.2
0
5
10
15
20
25
30
35
40
45
FloodRiskMapper
Flowroute
InfoWorks2D
ISIS2D
JFLOW-GPU
MIKE FLOOD
Dynamic RFSM
SOBEK
TUFLOW
UIM
Direct RFSM
ANUGA
TUFLOW FV

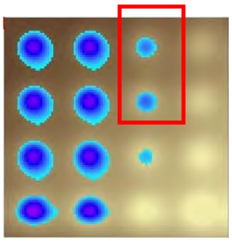

Test:2-Point:8
Water level (m)
Time (hours)
9.5
9.4
9.3
9.2
9.1
9
8.9
0
5
10
15
20
FloodRiskMapper
Flowroute
InfoWorks2D
ISIS2D
JFLOW-GPU
MIKE FLOOD
Dynamic RFSM
SOBEK
TUFLOW
UIM
Direct RFSM
ANUGA
TUFLOW FV

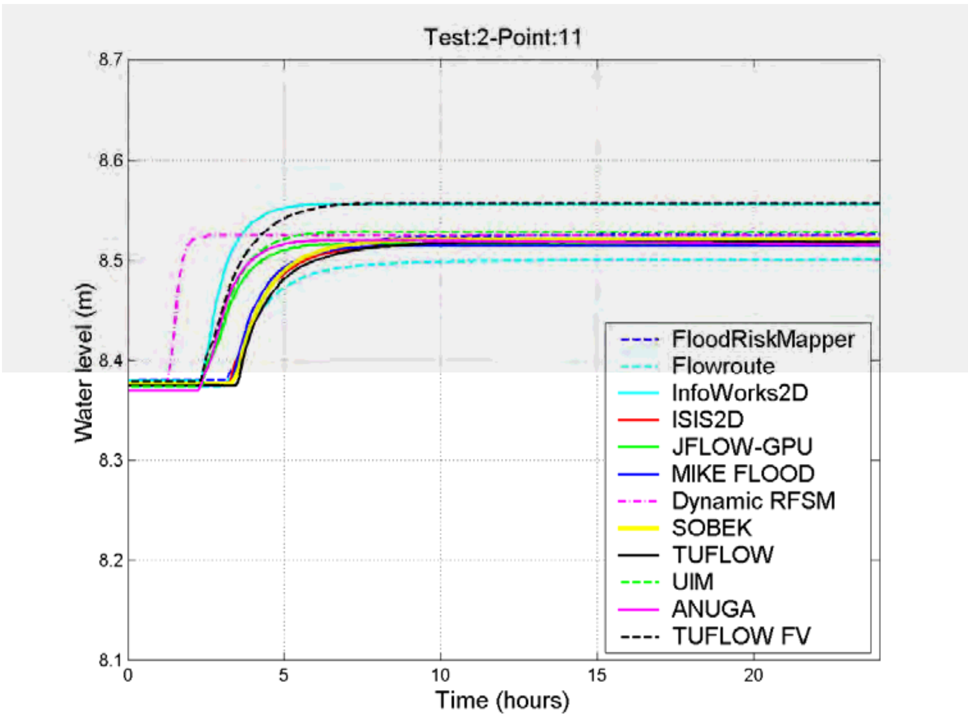

Test:2-Point:7
Water level (m)
Time (hours)
9.1
9
8.9
8.8
8.7
8.6
0
5
10
15
20
FloodRiskMapper
Flowroute
InfoWorks2D
ISIS2D
JFLOW-GPU
MIKE FLOOD
Dynamic RFSM
SOBEK
TUFLOW
UIM
Direct RFSM
ANUGA
TUFLOW FV

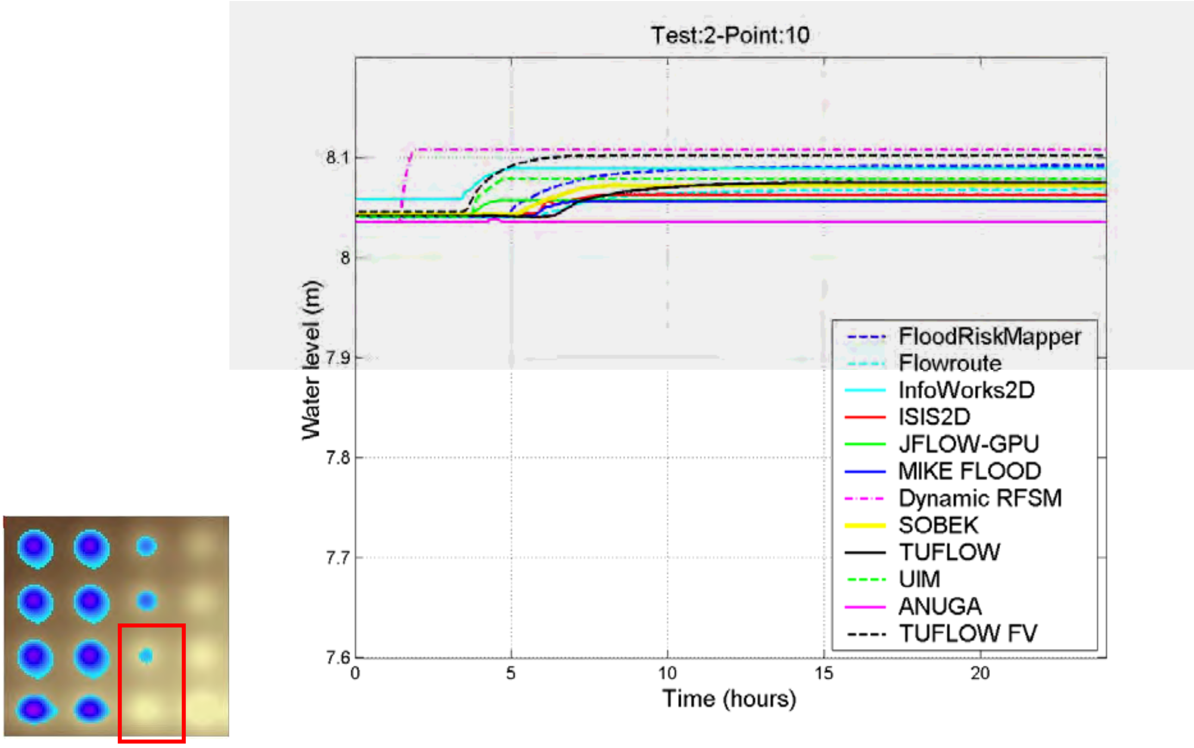

Test:2-Point:6
Water level (m)
Time (hours)
FloodRiskMapper
Flowroute
InfoWorks2D
ISIS2D
JFLOW-GPU
MIKE FLOOD
Dynamic RFSM
SOBEK
TUFLOW
UIM
Direct RFSM
ANUGA
TUFLOW FV

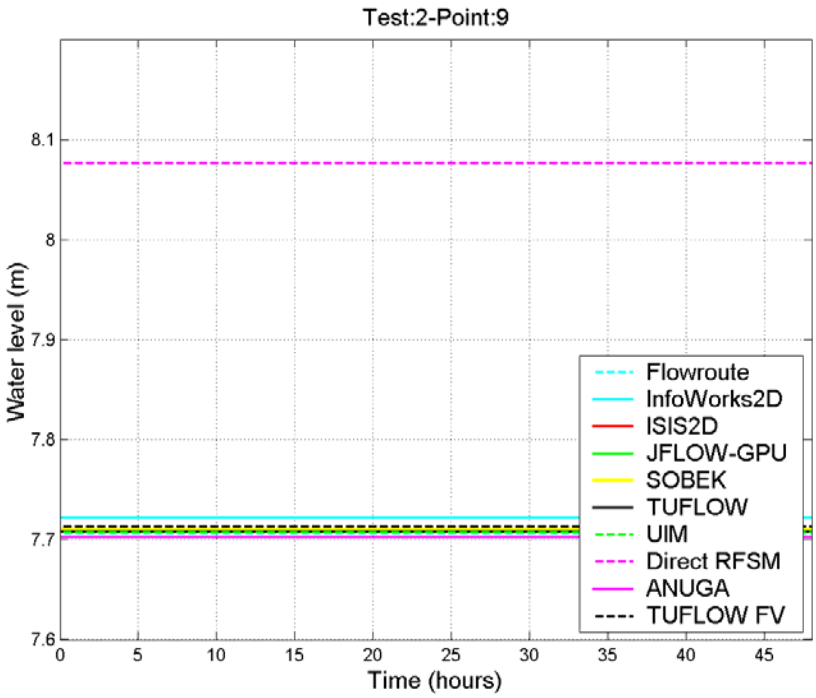

Test:2-Point:5
Water level (m)
Time (hours)
FloodRiskMapper
Flowroute
InfoWorks2D
ISIS2D
JFLOW-GPU
MIKE FLOOD
Dynamic RFSM
SOBEK
TUFLOW
UIM
Direct RFSM
ANUGA
TUFLOW FV

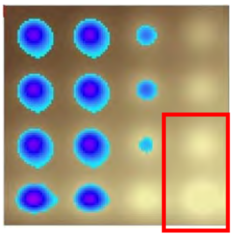

Test:2-Point:12
Water level (m)
Time (hours)
9
8.9
8.8
8.7
8.6
8.5
8.4
0
5
10
15
20
FloodRiskMapper
Flowroute
InfoWorks2D
ISIS2D
JFLOW-GPU
MIKE FLOOD
Dynamic RFSM
SOBEK
TUFLOW
UIM
ANUGA
TUFLOW FV

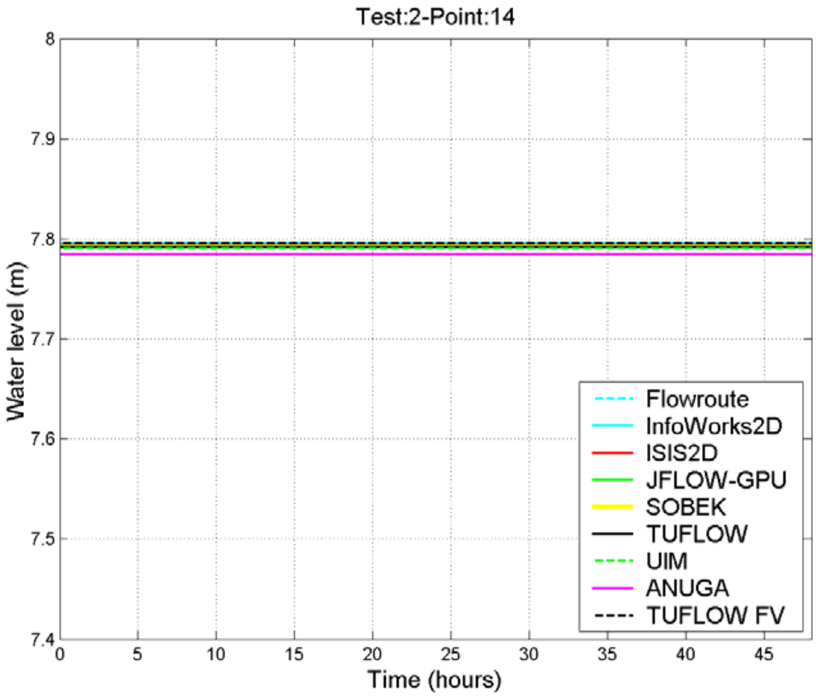


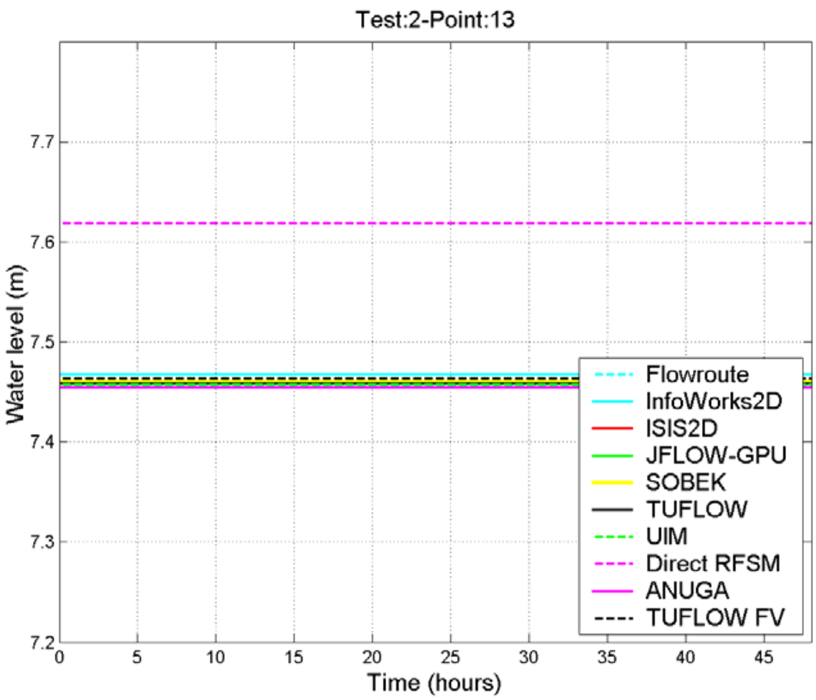

Test:2-Point:11
Water level (m)
Time (hours)
8.7
8.6
8.5
8.4
8.3
8.2
8.1
0
5
10
15
20
FloodRiskMapper
Flowroute
InfoWorks2D
ISIS2D
JFLOW-GPU
MIKE FLOOD
Dynamic RFSM
SOBEK
TUFLOW
UIM
ANUGA
TUFLOW FV


Test:2-Point:10
Water level (m)
Time (hours)
FloodRiskMapper
Flowroute
InfoWorks2D
ISIS2D
JFLOW-GPU
MIKE FLOOD
Dynamic RFSM
SOBEK
TUFLOW
UIM
ANUGA
TUFLOW FV


Test:2-Point:9
Water level (m)
Time (hours)
Flowroute
InfoWorks2D
ISIS2D
JFLOW-GPU
SOBEK
TUFLOW
UIM
Direct RFSM
ANUGA
TUFLOW FV







Full models:

The behaviour observed in the results by the full models is as follows. A transient water level peak is observed near the inflow while the inflow takes place. After the inflow has stopped the water level in every depression gradually decreases until it eventually reaches the level of the lowest ‘sill’ separating it from surrounding depressions (Points 1,2,3,4,6,7,8). Insufficient water has flowed to depressions 5,10,11,12 to fill them and at these points the water stabilises eventually at a level below the next ‘sill’. The transient peak level is ~ 20cm above the final level at point 4 (near the inflow), ~ 10cm at points 3 and 8, ~5 cm at points 2 and 7, becoming imperceptible or non-existent at points further away from the inflow. This transient peak lasts for approximately the duration of the inflow at point 4 near the inflow, and is more spread out in time at other points. Flood arrival times are similar between models initially, but discrepancies up to several hours can be observed at points further away from the inflow, most notably point 5 and point 10, with predictions from TUFLOW FV and InfoWorks 2D showing arrival times clearly ahead of the other packages. These differences can be attributed to a) the fact that the flow of water between the depressions is very shallow, which (as is often the case with shallow flows) can give rise to significant differences between models; b) differences in the implementation of the boundary condition. At points away from the inflow differences in timing occur as a result of the accumulation of differences occurring upstream.

Discrepancies in terms of the final level predicted by the full models are as follows:

- At points 1,2,3,4,6,7,8 the differences are small (<0.01m). This is within the limits expected due to topographic effects and wet/dry parameter settings.
- At point 5 the levels are still rising at the end of the simulation and comments are not possible
- At point 10 the differences are within a ~0.05m range, which considering the bowl shaped depression and the very shallow depths (<0.06m) corresponds to very small quantities of water.
- At point 11 all predictions are within a small ~0.015m range, except those by TUFLOW FV and InfoWorks 2D which were ~0.04m higher.
- At point 12 all predictions are within a small ~0.02m range, except those by TUFLOW FV which were very slightly higher (by ~0.02m).

Simplified models:

Flood Risk Mapper: There is evidence on the graphs for several points (2, 4, 8), that 'sill' elevations (between depressions) were created by the model to be several centimetres (~3cm) below those in other models. This is unexpected (differences in DEM elevations of ~3cm existed within a length scale at least as large as ~40m, ie. much more than the 10m grid resolution used by FRM). This causes concern regarding the way in which cell elevations were assigned in this Flood Risk Mapper model. Differences in timing and final levels at downstream points are explained at least partly by this effect.

Flowroute: The final level at points 2,3,4,6,7,8 is consistently 1 to 2 cm higher than that predicted by most full models, suggesting an unexpected topography effect, albeit smaller than in the case of Flood Risk Mapper. At downstream points 5,10,11,12 this resulted in later arrival times and slightly lower final levels (up to at least ~0.05m).

JFLOW-GPU: the results are consistently similar to those of the full models, with the exception of Point 5 where JFLOW-GPU predicted a markedly delayed and slower rise stabilising below the expected final levels predicted by the full models (by at least 0.1m).

RFSM (Dynamic): predicted flood arrival times are earlier and transient peak levels are lower than with other models. However final levels are generally consistent with the full models' predictions with the exception of at least Point 1, by ~0.07m, and possibly Point 5.

RFSM (Direct): the predicted (final) levels are different from other models' predictions by more than 0.1m at least at points 6, 9, 11, 12, 13. Water is predicted to flow along the bottom row of depressions to points 9 and 13, where other models do not predict any inundation. Such discrepancies are not unexpected considering the simplified representation of physics applied by the RFSM. Final inundation extent in this test is to a significant extent governed by conservation of momentum, while the RFSM (Direct) is only appropriate for problems where gravity is by far the dominant physical process (ie. inertia and resistance to flow are small).

Volume conservation

The largest volume change reported is a 1.4% volume loss by MIKE FLOOD. This did not have any identifiable consequence in the results, and the effect of model choice was clearly more significant than a lack of volume conservation of this magnitude.

4.2.3 Conclusion and discussion from Test 2

All **full models** predicted very similar results in terms of the final inundation extent (i.e. final levels). Differences between model level predictions were small and within the level of acceptable accuracy for practical application to a problem of this type.

The high level of consistency in the results of the full models provides ground for a high level of confidence in the accuracy of these results.

The **simplified models** tended to provide noticeably more markedly different results (final level differences compared to full models in excess of ~0.05m at several locations), however these may also be considered acceptable depending on the level of accuracy required.

However the RFSM Direct predicted a significantly different final inundation extent[9].

[9] It is acknowledged that the model's run time in this test was at least 10 times shorter than the RFSM Dynamic and 30 times faster than any other model.

4.2.4 Summary of relevant technical information

TESTS 2 (1) Name	(2) Version (+ numeric. scheme)	(3) Hardware	(4) Multi-Proc.	(5) Grid (20m or 10000 elem.)	(6) Time-stepping	(7) Run time (min)	(8) Final volume (m^3)
ANUGA	1.1beta_7501	Intel Mobile Core 2 Duo T7500 (Merom) 2.2 GHz RAM 2048MB (DDR2)	no	10088 elements	adaptive	18.8	97223.15
Flood Risk Mapper	FRM 0.26	Intel® Core™ Duo, T2500 @ 2.00Ghz, 1GB of RAM	no	10m	adaptive	18	97200
FloodFlow	W.12.0 Beta ADI	Intel(R) Core™ 2 CPU 6400 2.13GHz RAM 2GB	no	20m	adaptive	130	97200
Flowroute	2.9.8	2.4Ghz (Intel Q6600) RAM 4GB	OMP	20m	0.08s	24	97200
InfoWorks	RS 2D v10.5	Intel® Core™ i7 920 2.66GHz Quad Core 12 GB RAM (DDR3-1333)	OMP	9997 triangles	adaptive	0.73	97203.11
ISIS	3.2.0.21 ADI	Quad Intel Xeon DP 5050 @ 3.0 GHz, 4096MB RAM (FB-DDR2)	Partial, see section 4.0.3	20m	10s	1.58	97300
JFLOW-GPU	JFLOW-GPU DW	AMD Phenom II X4 940 3.0 GHz RAM 2.25 GB GPU: NVIDIA GeForce GTX 295	Yes - GPU	20m	adaptive average 0.997s	1.83	97198.4
MIKE FLOOD	2009 incl. Serv. Pack 2	Intel Core 2 Duo CPU P8600 2.4 GHz RAM 4.00 GB	no	20m	10s	0.40	95851
RFSM (Direct)	*3.5.4*	Intel Dual Xeon 2 cores of 3GHz RAM 2GB	no	16 elements[10]	N/A	0.02	97200
RFSM (Dynamic)	0.1 (Beta)	Intel Dual Xeon 2 cores of 3GHz RAM 2GB	no	16 elements[11]	100s	0.19	97200
SOBEK	2.13	Intel i7 8 core CPU 2.66 GHz	no	20m	15s	1.67	97200
TUFLOW	2010-01-AD-iDP	Intel Core 2 Duo T9800 2.93GHz RAM 4Gb	no	20m	10s	1.92	97069
TUFLOW FV	0.107.0006 2nd order solution	Intel Xeon X5472 3.00GHz RAM 8Gb	Yes – 8 CPUs	20m	Typical time step: 1.5s	2.64 (0.60)	97200

[10] See Appendix B.6.
[11] See Appendix B.6.

TESTS 2 (1) Name	(2) Version (+ numeric. scheme)	(3) Hardware	(4) Multi-Proc.	(5) Grid (20m or 10000 elem.)	(6) Time-stepping	(7) Run time (min)	(8) Final volume (m^3)
UIM	2009.12	Dual Quad-core 2.83GHz Intel Xeon E5440 Harpertown node RAM 16GB	OMP	20m	1s	11.87	97200

Reasons for undertaking Test 2 at a different grid resolution.

Flood Risk Mapper: “FRM does not have the capability to run a 20m grid”

Heriot Watt University Response: this is surprising given that the resolution of the DEM was 2m.

Other information provided:

RFSM (Dynamic): The “Impact Zones” (computational elements) were automatically defined from the topography. A weir relation was used for the discharge calculation at the interfaces between IZs. The weir coefficient was set to 0.35.

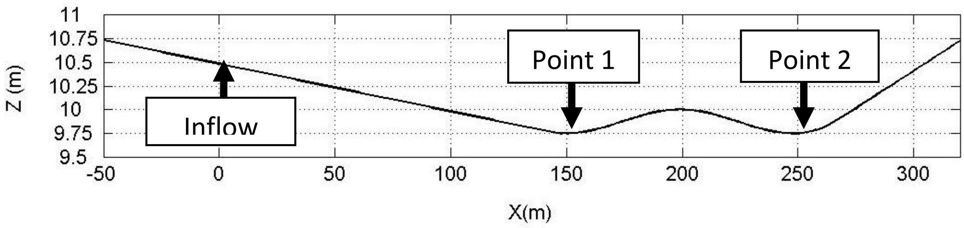
Heriot Watt University Response: it would be useful to know how this value of the weir coefficient is set.

4.3 Test 3: Momentum conservation over a small obstruction

4.3.1 Introduction

This test (see Appendix A.3 for details) consists of a sloping topography with two depressions separated by an obstruction as illustrated in Figure 3, and of width 100m. A varying inflow discharge is applied as an upstream boundary condition at the left-hand end, causing a flood wave to travel down the 1:200 slope. While the total inflow volume is just sufficient to fill the left-hand side depression at x=150m, some of this volume is expected to overtop the obstruction because of momentum conservation and settle in the depression on the right-hand side at x=250m.

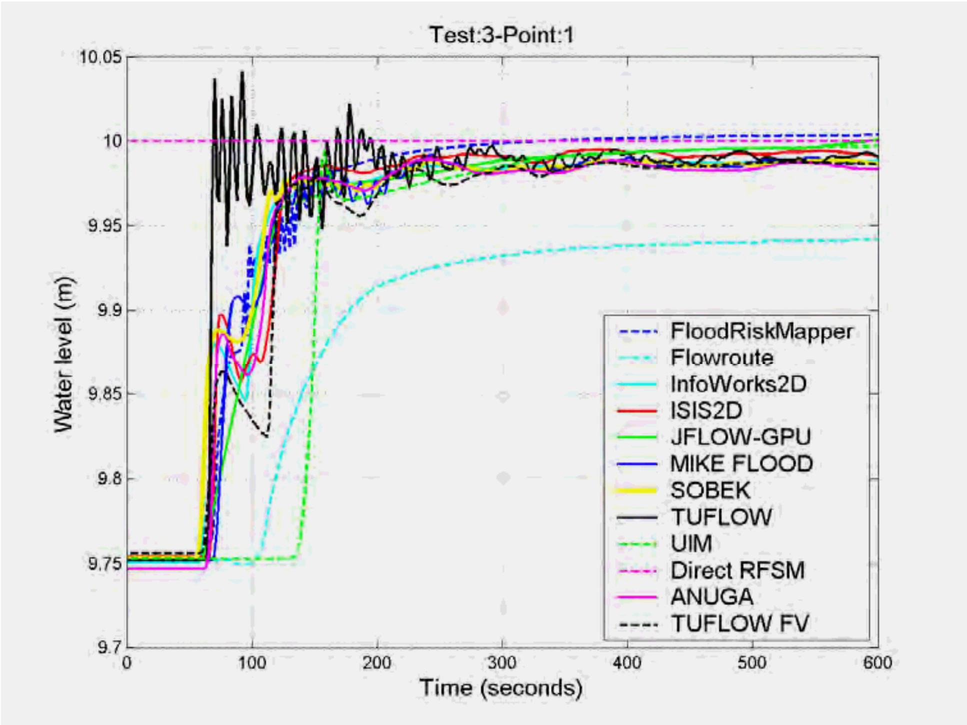
Figure 3: Profile of DEM used in Test 3

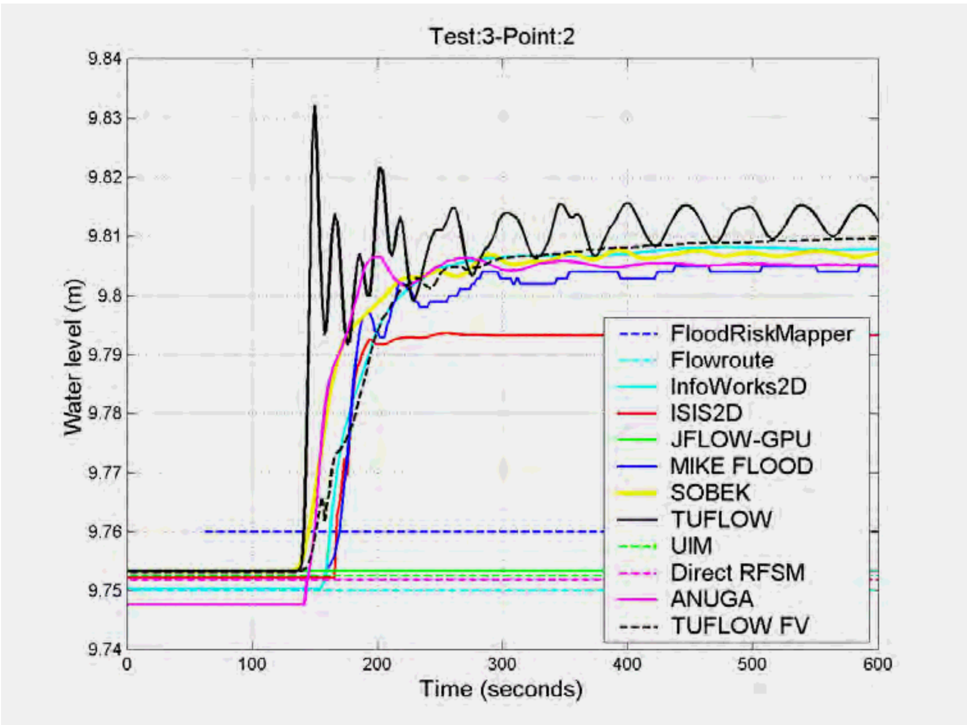


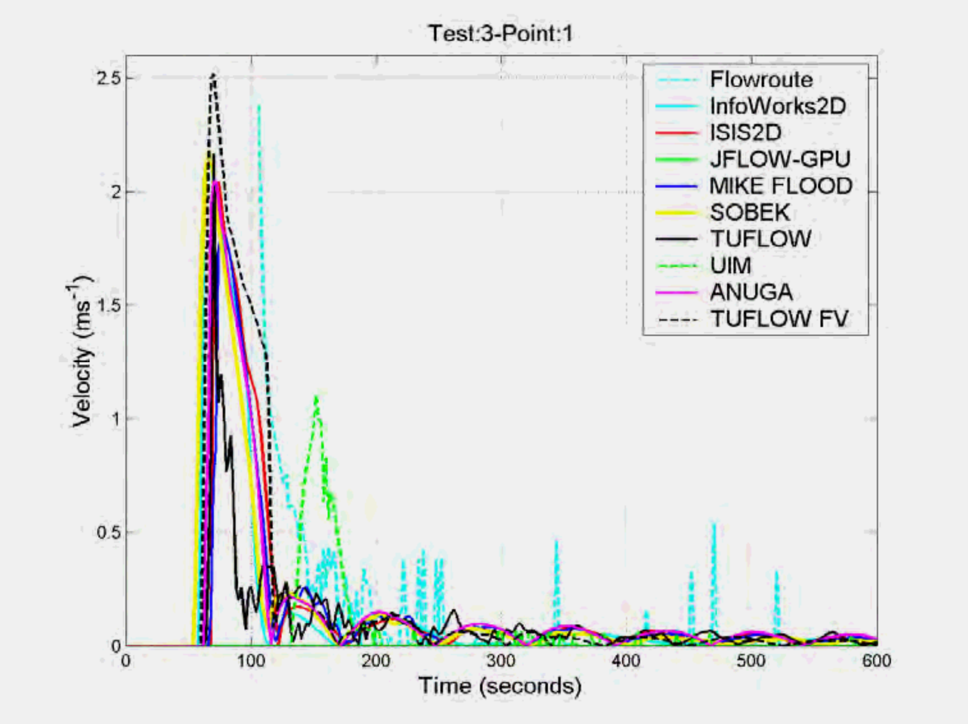
The objective of the test is to assess each package's ability to conserve momentum over an obstruction in the topography.

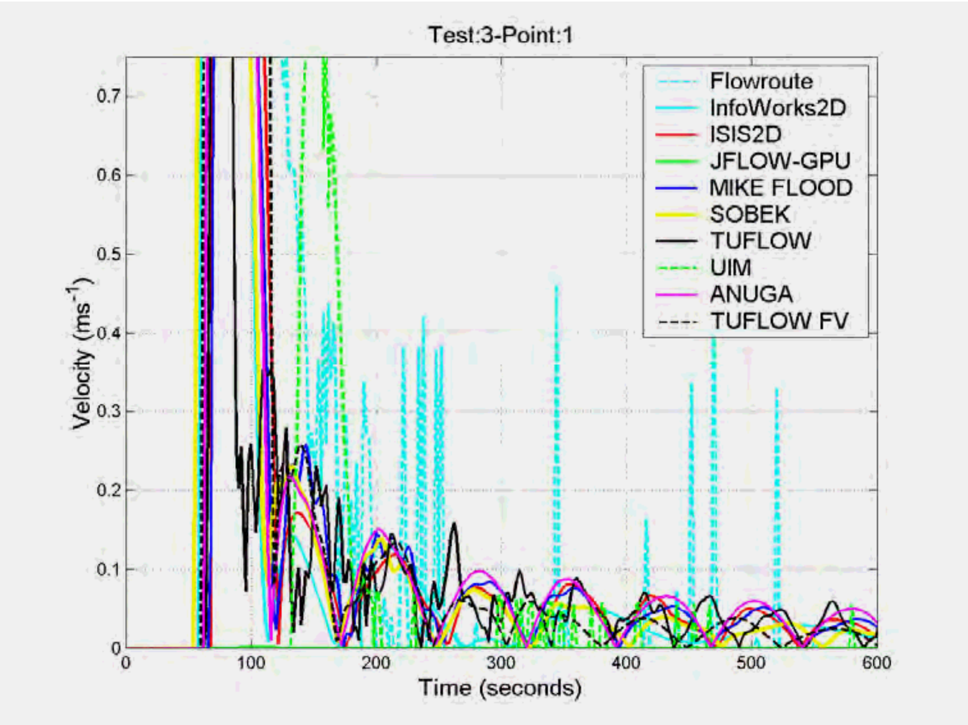
Although no exact solution exists for Test 3, any model relying on the full shallow water equations (i.e. momentum conservation including the acceleration terms) is expected to predict a water level rise at Point 2.

4.3.2 Water level and velocity time series









Note: the figure above (Velocity at Point 1) is a close-up of the previous figure.

Full models:

At point 1 most full models predicted a rapid increase of the water level from height 9.75m at t≈60s to height ~9.98/9.99m at t≈120s, including a short-lived recession of a few cms at mid-height (with the exception of TUFLOW which predicted a more rapid rise with subsequent oscillatory behaviour, most likely due to lack of shock capturing functionality[12]). Following this the level became gradually closer to 10m over a few minutes. After t≈150s, the level rose quickly by ~5cm to ~6cm at point 2 on the other side of the obstruction. Differences between models in this may be partly due to the treatment of shocks on the left-hand side of the obstruction. Velocities showed a rapid rise and fall as the flood wave passed point 1. The curves then reflect the slow oscillations of the water back and forth in the first depression with a period ~150s.

Simplified models:

As expected, none of the simplified models (Flood Risk Mapper, Flowroute, JFLOW-GPU,Direct RFSM, UIM) predicted any water to flow over the obstruction. The behaviour on the left hand side of the obstruction (Point 1) was broadly similar to that of the full models for Flood Risk Mapper, JFLOW-GPU and UIM (although delayed for the latter). The final level reached at point 1 according to Flowroute is evidence of a significantly insufficient volume (either due to numerical loss or to an incorrect boundary condition).

The simplified models did not predict any meaningful velocities at Point 1.

4.3.3 Conclusions and discussion from Test 3

All **full models** predicted very similar results, mainly in that water flowed over the obstruction because of momentum conservation, with similar depths at Point 2.

As expected none of the **simplified models** predicted any water to flow over the obstruction.

[12] As confirmed by WBMBMT.

4.3.4 Summary of relevant technical information

TEST 3 / (1) Name	(2) Version (+ numeric. scheme)	(3) Hardware	(4) Multi-Proc.	(5) Grid (5m or 1200 elements)	(6) Time-stepping	(7) Run time (s)
ANUGA	1.1beta_7501	Intel Mobile Core 2 Duo T7500 (Merom) 2.2 GHz RAM 2048 MB (DDR2)	no	1207 elements	adaptive < 1s	6
Flood Risk Mapper	FRM 0.26	Intel® Core™ Duo, T2500 @ 2.00Ghz, 1GB of RAM	*no*	5m	Adaptive: <2s	10
FloodFlow	Not tested					
Flowroute	2.9.8	2.4Ghz (Intel Q6600) RAM 4GB	OMP	5m	0.04s	74
InfoWorks	RS 2D v10.5	Intel® Core™ i7 920 2.66GHz Quad Core 12 GB RAM (DDR3-1333)	OMP	1210 triangles	2s	10
ISIS	3.2.0.21 TVD	Intel Core 2-Quad CPU Q6600 2.4 GHz RAM 2.0 GB	no	5m	0.25s	23
JFLOW-GPU	JFLOW-GPU DW	AMD Phenom II X4 940 3.0 GHz RAM 2.25 GB GPU: NVIDIA GeForce GTX 295	Yes - GPU	5m	Avg 0.081s	27.4
MIKE FLOOD	2009 incl. Serv. Pack 2	Intel Core 2 Duo CPU P8600 2.4 GHz RAM 4.00 GB	no	5m	1s	1
RFSM (Direct)	*3.5.4*	Intel Dual Xeon 2 cores of 3GHz RAM 2GB	no	2 elements[13]	N/A	<1s
RFSM (Dynamic)	*Not tested*					
SOBEK	2.13	Intel i7 8 core CPU 2.66 GHz	no	5m	0.1s	20
TUFLOW	2010-01-AD-iSP	Intel Core 2 Duo T9800 2.93GHz RAM 4Gb	no	5m	3s	1.8
TUFLOW FV	0.107.0006 2nd order solution	Intel Xeon X5472 3.00GHz RAM 8Gb	Yes – 8 CPUs	5m	Typical time step: 0.18s-0.2s	2.93 (0.94)
UIM	2009.12	Intel Core™ 2 Duo CPU T7800 2.60GHz RAM 3GB	OMP	5m	0.01s	56s

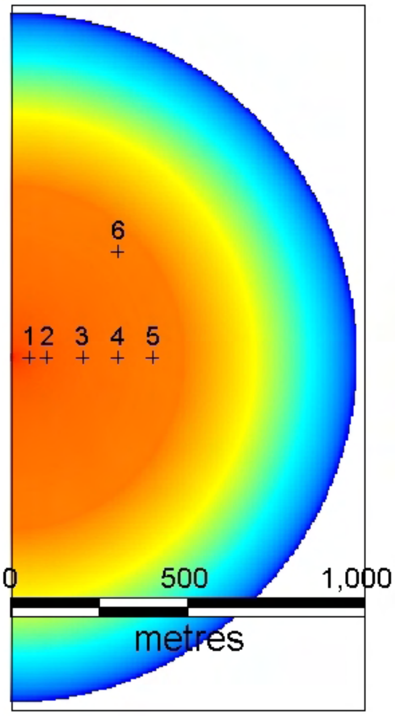
[13]See Appendix B.6.

4.4 Test 4: Speed of flood propagation over an extended floodplain

4.4.1 Introduction

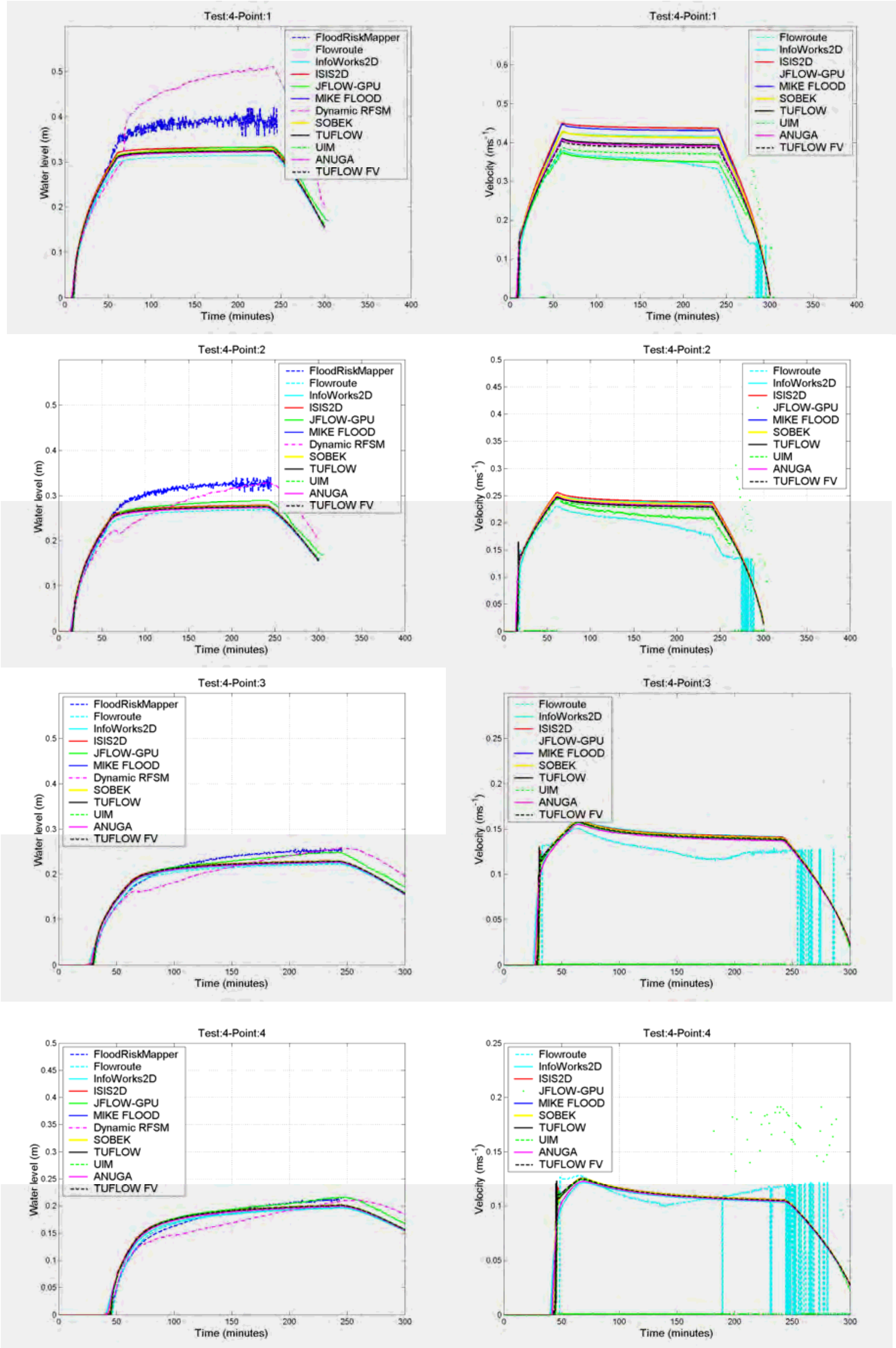
The test (see Appendix A.4 for details) consists of a flat horizontal floodplain of dimensions 1000m x 2000m, with a single inflow boundary condition, simulating the failure of an embankment by breaching or overtopping, with a peak flow of 20 m^3/s and time base of ~ 5 hours. The boundary condition is applied along a 20m line in the middle of the western side of the floodplain.

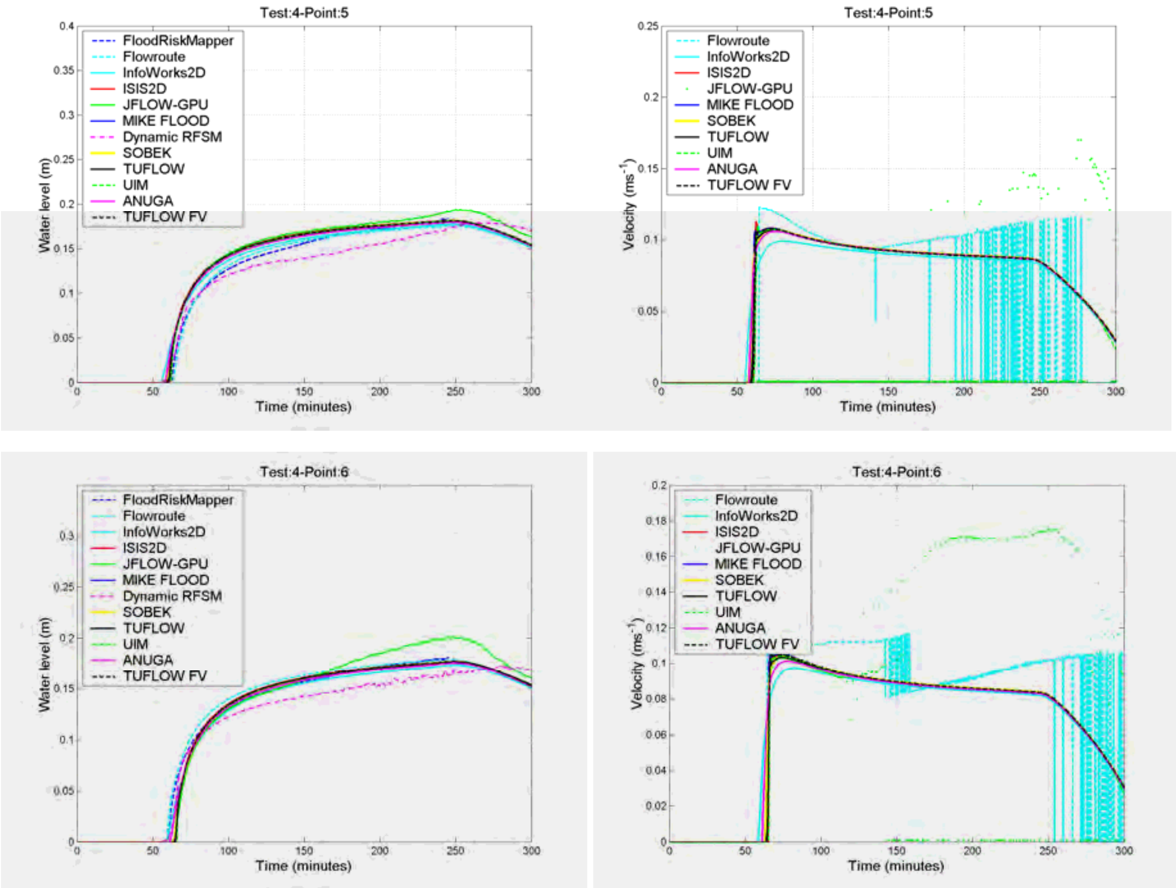
Figure 4: Location of output points, with a typical flood distribution at time 3 hours.



The objective of the test is to assess the package's ability to simulate the celerity of propagation of a flood wave and predict transient velocities and depths. It is relevant to fluvial and coastal inundation resulting from breached embankments.

4.4.2 Water level (depth) and velocity time series


Test:4-Point:1
Water level (m)
Time (minutes)
FloodRiskMapper
Flowroute
InfoWorks2D
ISIS2D
JFLOW-GPU
MIKE FLOOD
Dynamic RFSM
SOBEK
TUFLOW
UIM
ANUGA
TUFLOW FV
Test:4-Point:1
Velocity (ms-1)
Time (minutes)
Flowroute
InfoWorks2D
ISIS2D
JFLOW-GPU
MIKE FLOOD
SOBEK
TUFLOW
UIM
ANUGA
TUFLOW FV
Test:4-Point:2
Water level (m)
Time (minutes)
Test:4-Point:2
Velocity (ms-1)
Time (minutes)
Test:4-Point:3
Water level (m)
Time (minutes)
Test:4-Point:3
Velocity (ms-1)
Time (minutes)
Test:4-Point:4
Water level (m)
Time (minutes)
Test:4-Point:4
Velocity (ms-1)
Time (minutes)



Full models

It can be observed on the figures above that most "full" models predicted *depths* and *velocities* at most output points within a few % of each other during the entire duration of the event, except for the initial rise of the flood at each output point, where arrival times were at most (points 5&6) within ~5min of each other (compared to a travel time of ~1hr), and where discrepancies only concerned very shallow flows.

Exceptions from this general behaviour only concern velocity predictions, as follows:

- At point 1 near the source differences between models were up to 20-25%. This may be due to differences in the approaches used to implement the boundary condition.
- Some FD models (including ISIS and TUFLOW) predict an initial sharp peak in the velocity prediction, reflecting the lack of shock-capturing properties.

Simplified models

Water levels:

Flowroute: Flowroute's peak depth predictions are consistently within a few % with the predictions by the full models.

Flood Risk Mapper: some of the predicted depths are oscillatory, with differences up to ~25% (near the inflow) compared to the full models.

JFLOW-GPU: predicted depths are similar to the predictions by the full models, with however small differences (up to 2.5cm, ~10%) towards the end of the event at the furthest away point (mostly 5&6).

RFSM (dynamic): predicted depths were generally within 10 to 20% of those of the full models, except at point 1 (nearest to the source) where they were up to ~50% larger.

UIM: UIM's water level predictions were generally within the range of predictions by the full models.

Velocities:

Flowroute and JFLOW-GPU: made some velocity predictions similar to the full models, but also showed large oscillations particularly away from the inflow and towards the end of the event.

UIM: predicted velocities consistently similar to predictions by the full models.

No time series velocity predictions were available from the Flood Risk Mapper or RFSM models.

4.4.3 Output in raster format

Figure 5: 0.15m depth contours at times 1hr (left) and 3hr (right). The colour coding is consistent with the one used in the rest of this report. Cross-sections shown in the two following figures were taken along the black dashed line, which starts at the left boundary and runs through output points 1 to 5.

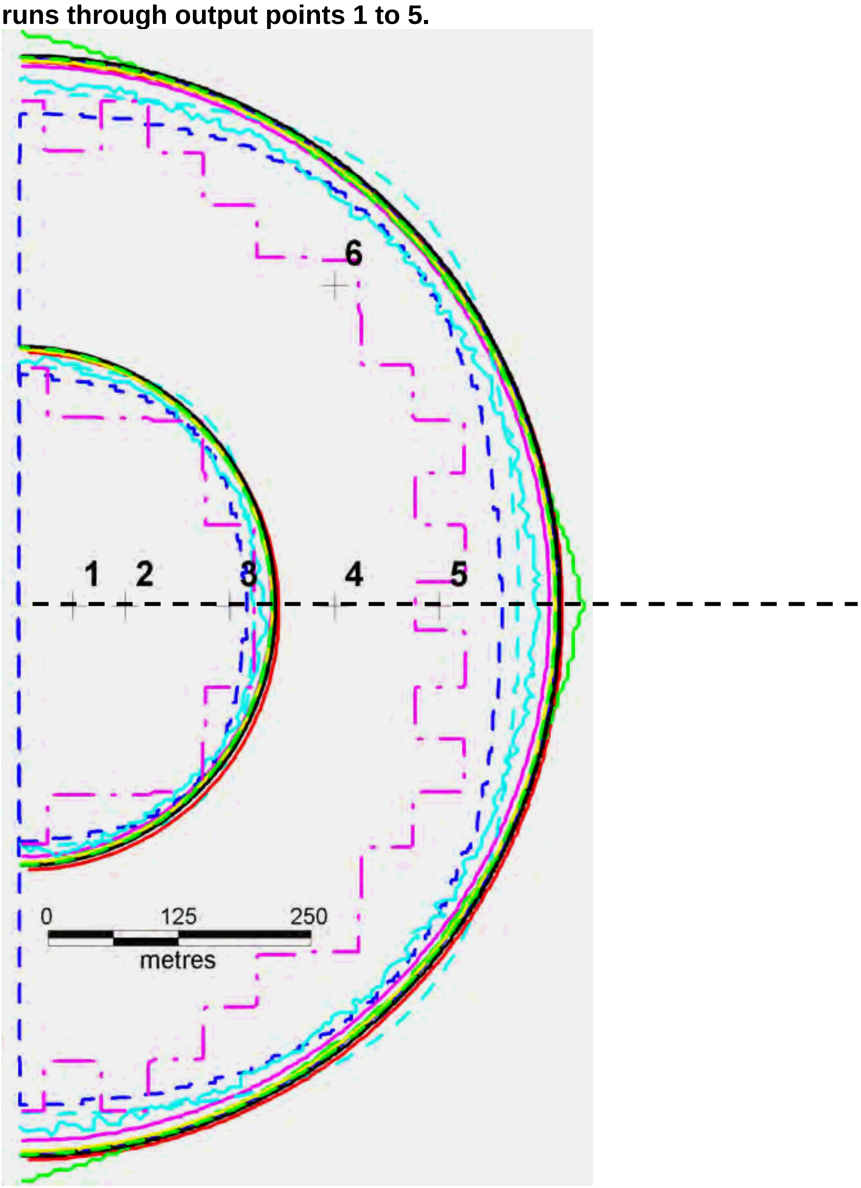


Figure 6: Cross-section of depths along the dashed line in previous figure, at time t=1hr.

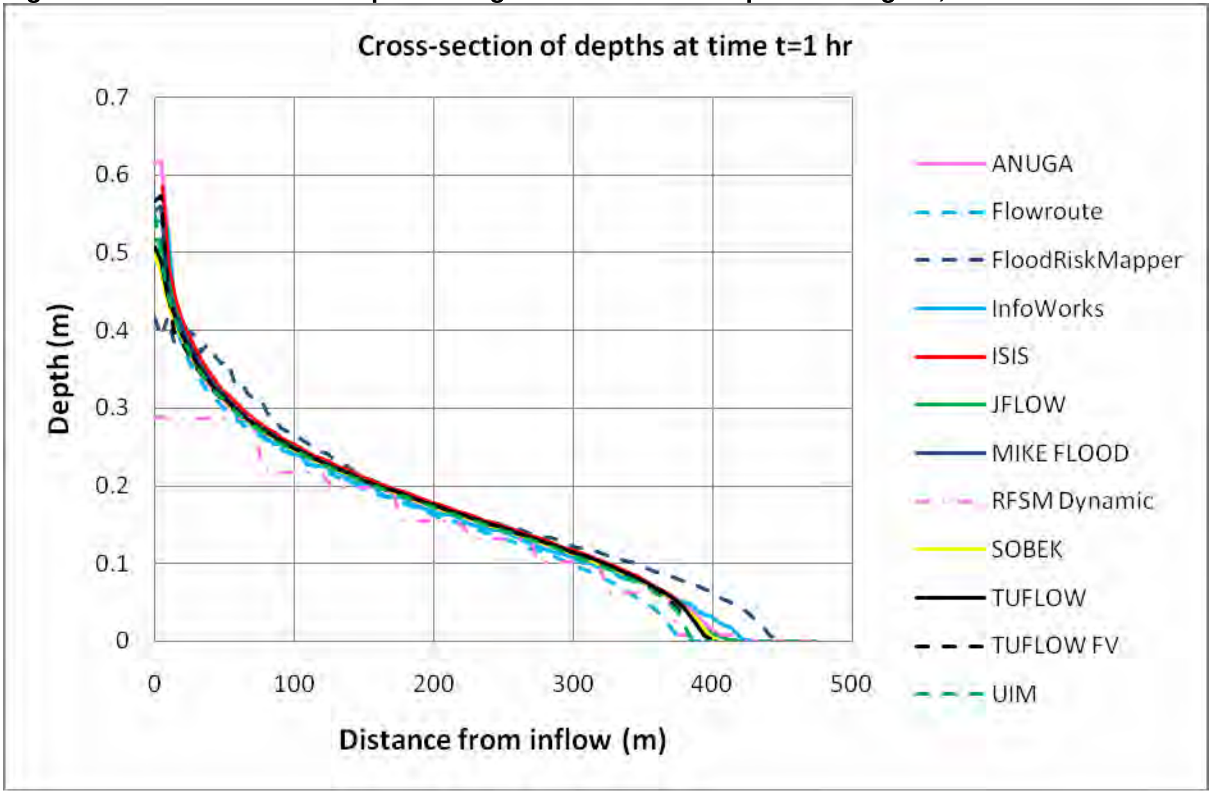
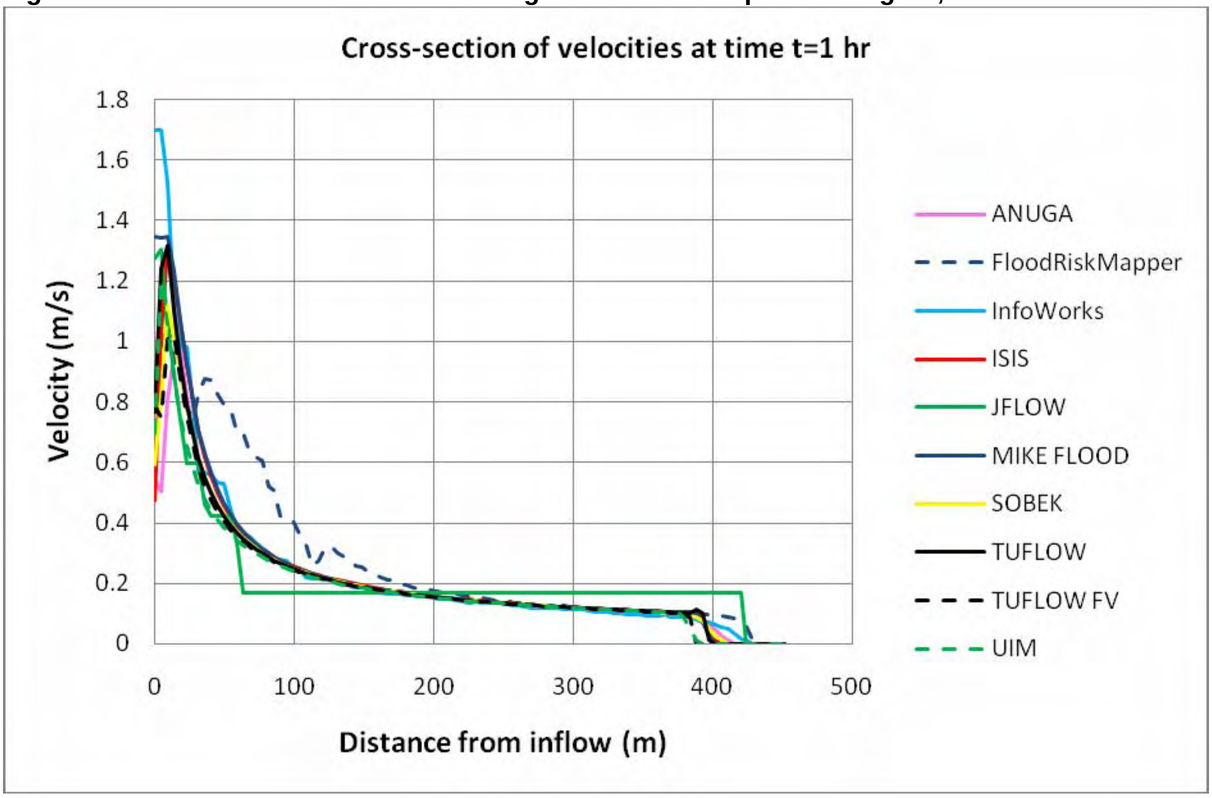


Figure 7: Cross-section of velocities along same line as in previous figure, at time t=1hr.



Although velocity results from Flood Risk Mapper were not provided as time series, they were provided in raster format and are illustrated in Figure 7. This shows velocities overestimated by up to ~80% where almost all other models are in agreement.

The stepped nature of the JFLOW-GPU curve is due to post-processing of the oscillatory results[14].

Observations from Figure 5, Figure 6 and Figure 7 are otherwise consistent with those from the time series in Section 4.4.2, particularly with regard to the results obtained for the simplified models, i.e. Flood Risk Mapper, Flowroute, JFLOW-GPU and RFSM Dynamic. They also show large discrepancies in the velocity predictions (including those by the full models) in the immediate vicinity of the inflow, due to differences in the implementation of the boundary.

4.4.4 Conclusions from Test 4.

All **full models** predicted very similar results in terms of travel times, peak water levels and peak velocities. Discrepancies between models were relatively small: ~10% for travel times, a few % for peak levels and velocities. This is unlikely to be larger than typical accuracy expectations in a problem of this type in a practical application.

The high level of consistency in the results of the full models provides ground for a high level of confidence in the accuracy of these predictions. It also suggests than in practical applications of flow modelling in the vicinity of a breach, topography effects (which are non-existent in Test 4 due to the perfectly horizontal ground) are likely to be more significant than differences in the numerical solution of the full shallow water equations.

However predictions of velocities in the immediate vicinity of the inflow is less accurate and depends on the set-up of the boundary condition (which was specified as a discharge vs. time as can be considered normal in a practical application).

The **simplified models** tended to provide markedly different results which, as far as *water levels* are concerned, may also be considered acceptable depending on the level of accuracy required. Flood Risk Mapper and RFSM Dynamic predictions deviate significantly from the predictions by other models in the immediate vicinity of the inflow.

Flowroute and JFLOW-GPU of velocity predictions oscillate significantly. The UIM predictions of velocity are in line with those of the full models[15]. Velocity predictions by Flood Risk Mapper and by RFSM are at the present time not available and were therefore not provided for this test[16].

Before drawing wider conclusions, it should be noted that this test case is more akin to a test of spreading rather than a test of the propagation of a rapidly advancing wave. It is likely that differences in performance between the full models and the simplified models would be larger in a case involving a rapidly advancing wave.

[14] As indicated by JBA: "The velocities obtained from JFLOW-GPU are considered indicative and should be viewed as such. In the case of Test 4 where velocity grids at specified times are required, the velocity grids have been reclassified."

[15] However a previous set of results from UIM, based on shorter time steps, had significant oscillations.

[16] Although velocity predictions by Flood Risk Mapper were provided in gridded format and show significantly overestimated velocities.

4.4.5 Summary of relevant technical information

TEST 4 (1) Name	(2) Version (+ numeric. scheme)	(3) Hardware	(4) Multi-Proc.	(5) Grid (5m or 80000 elements)	(6) Time-stepping	(7) Run time (min)
ANUGA	1.1beta_7501	Intel Mobile Core 2 Duo T7500 (Merom) 2.2 GHz RAM 2048 MB (DDR2)	no	80149 Elements	adaptive	60.8
Flood Risk Mapper	FRM 0.26	Intel® Core™ Duo, T2500 @ 2.00Ghz, 1GB of RAM	no	5m	Adaptive <20s	33[17]
FloodFlow	W.12.0 Beta ADI	Intel(R) Core™ 2 CPU 6400 2.13GHz RAM 2GB	no	5m	Adaptive	75
Flowroute	2.9.8	2.4Ghz (Intel Q6600) RAM 4GB	OMP	5m	0.01s	92
InfoWorks	RS 2D v10.5	Intel® Core™ i7 920 2.66GHz Quad Core 12 GB RAM (DDR3-1333)	OMP	79857 Triangles	Adaptive	6.5
ISIS	3.2.0.21 ADI	Intel Core 2-Quad CPU Q6600 2.4 GHz RAM 2.0 GB	Partial, see section 4.0.3	5m	5s	28.7
JFLOW-GPU	JFLOW-GPU DW	AMD Phenom II X4 940 3.0 GHz RAM 2.25 GB GPU: NVIDIA GeForce GTX 295	Yes - GPU	5m	Avg 0.1811s	2.3
MIKE FLOOD	2009 incl. Serv. Pack 2	Intel Core 2 Duo CPU P8600 2.4 GHz RAM 4.00 GB	no	5m	5s	1.27
RFSM (Direct)	*Not tested*					
RFSM (Dynamic)	0.1 (Beta)	Intel Dual Xeon 2 cores of 3GHz RAM 2GB	no	861 elements[18]	15s	5.8
SOBEK	2.13	Intel i7 8 core CPU 2.66 GHz	no	5m	2s	16.9
TUFLOW	2010-01-AD-iSP	Intel Core 2 Duo T9800 2.93GHz RAM 4Gb	no	5m	5s	5.1
TUFLOW FV	0.107.0006 2nd order solution	Intel Xeon X5472 3.00GHz RAM 8Gb	Yes – 8 CPUs	5m	Typical time step: 0.2 - 0.4s	24.5 (5.0)
UIM	2009.12	Dual Quad-core 2.83GHz Intel Xeon E5440 Harpertown node RAM 16GB	OMP	5m	0.1s	282.8

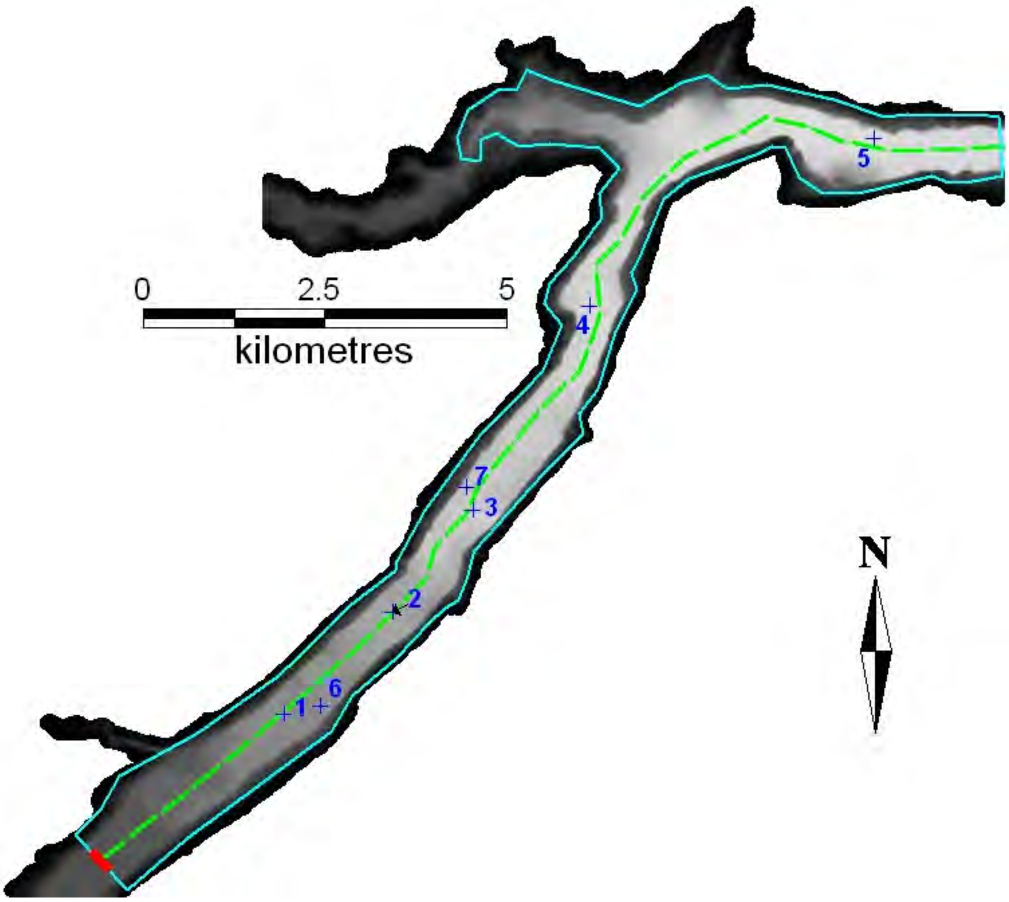
[17] Modified to account for early end of model.
[18] See Appendix B.6.

4.5 Test 5: Valley flooding

4.5.1 Introduction

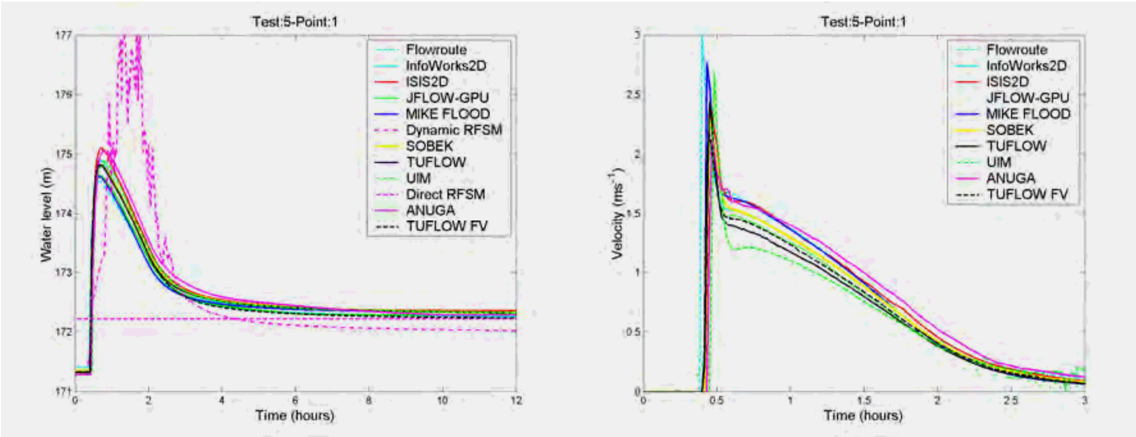
This test (see Appendix A.5 for details) is designed to simulate flood wave propagation down a river valley following the failure of a dam, represented by a skewed trapezoidal inflow hydrograph with a short early peak at 3000 m^3/s. The valley is represented in Figure 8.

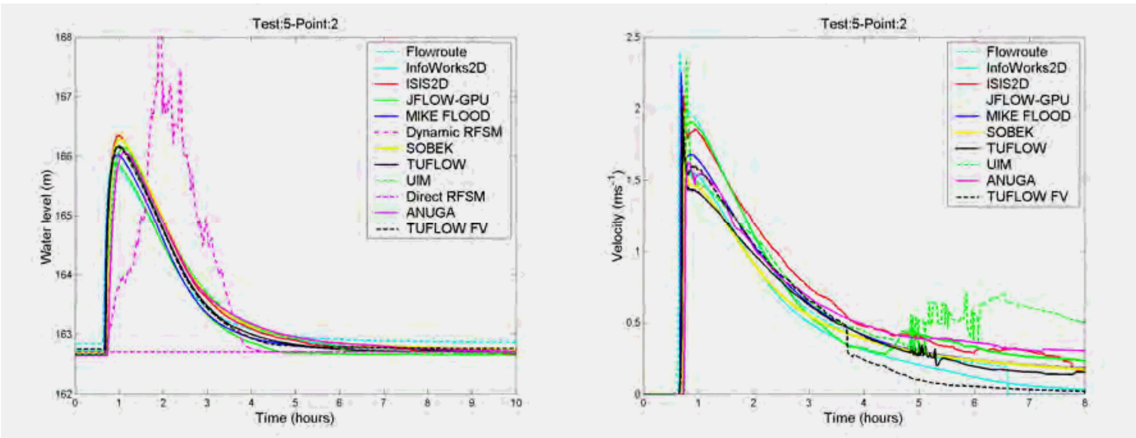
Figure 8: Map of the valley used in Test 5, with the inflow at the red line and 7 output points.

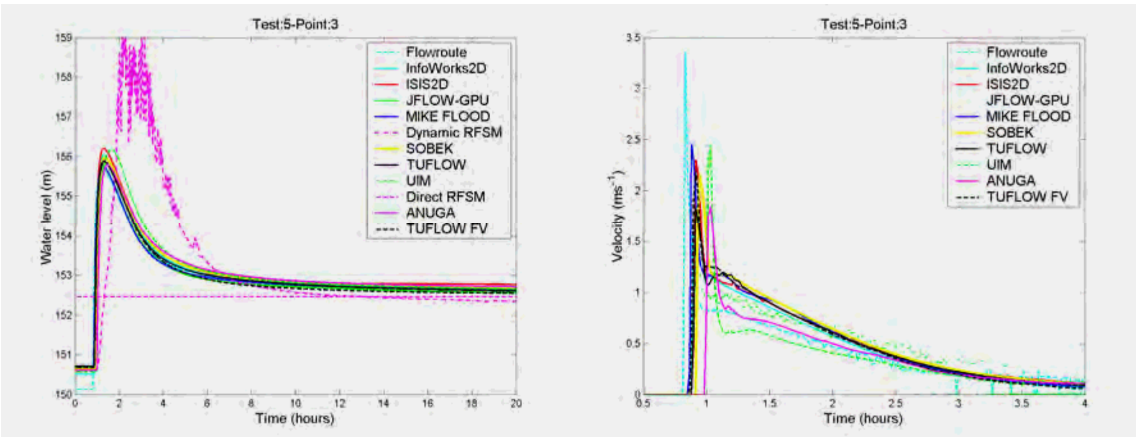


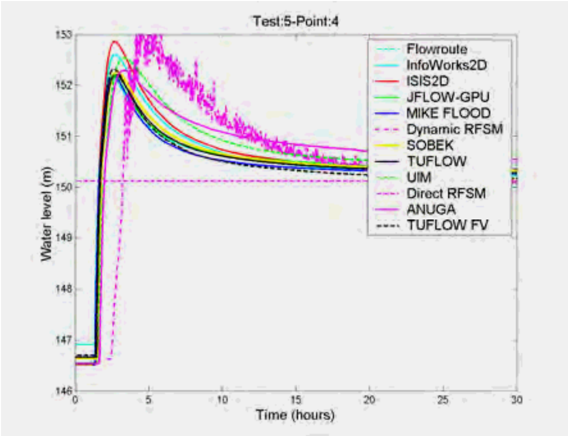
The objective of the test is to assess the package's ability to to simulate major flood inundation and predict flood hazard arising from dam failure (peak levels, velocities, and travel times).

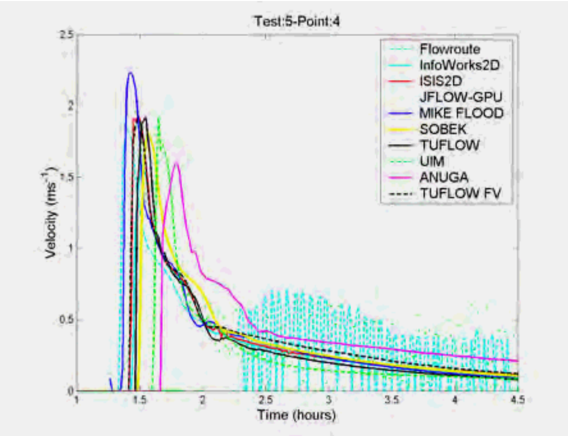
4.5.2 Water level and velocity time series

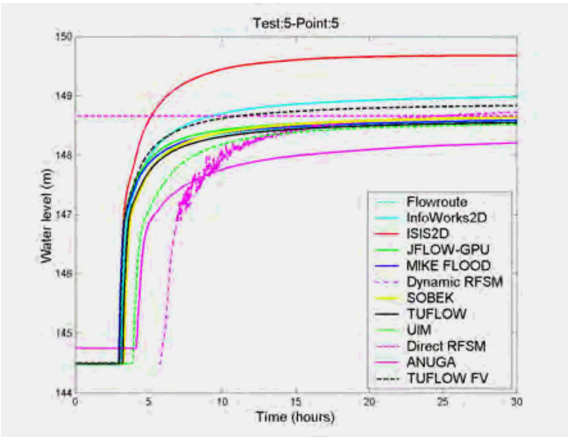


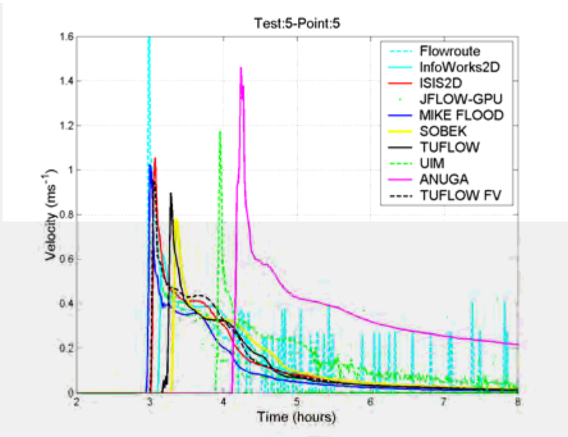


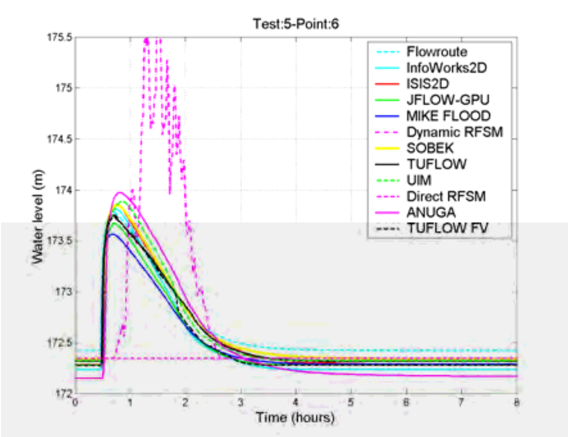


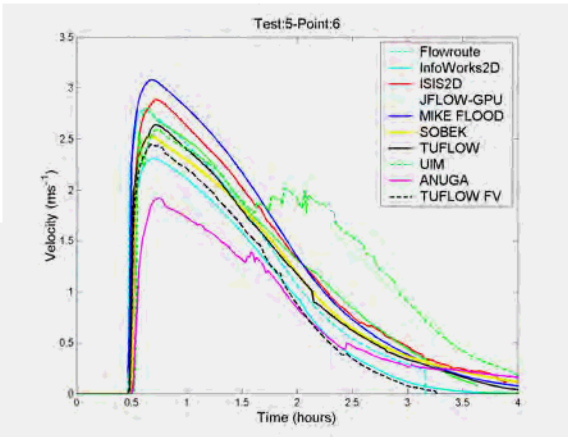

Test:5-Point:4
Water level (m)
Time (hours)
Flowroute
InfoWorks2D
ISIS2D
JFLOW-GPU
MIKE FLOOD
Dynamic RFSM
SOBEK
TUFLOW
UIM
Direct RFSM
ANUGA
TUFLOW FV

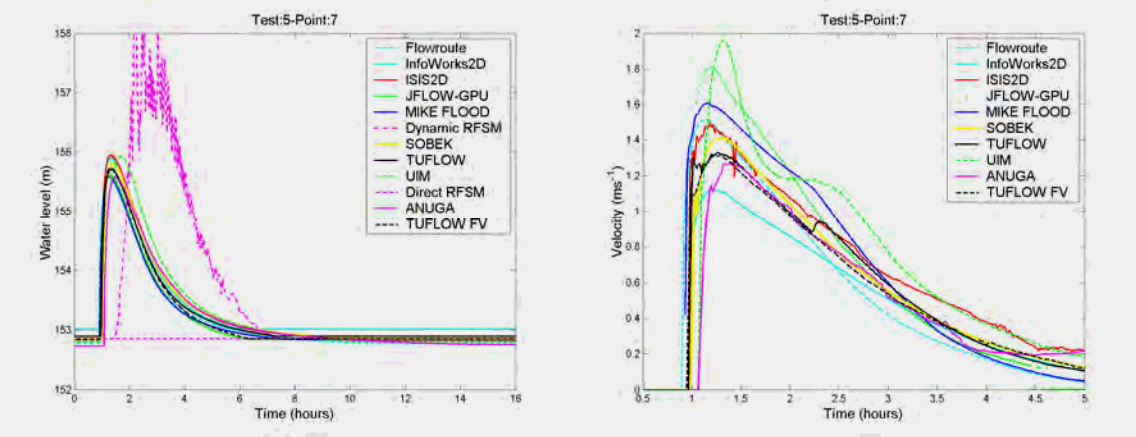

Test:5-Point:4
Velocity (ms^{-1})
Time (hours)
Flowroute
InfoWorks2D
ISIS2D
JFLOW-GPU
MIKE FLOOD
SOBEK
TUFLOW
UIM
ANUGA
TUFLOW FV


Test:5-Point:5
Water level (m)
Time (hours)
Flowroute
InfoWorks2D
ISIS2D
JFLOW-GPU
MIKE FLOOD
Dynamic RFSM
SOBEK
TUFLOW
UIM
Direct RFSM
ANUGA
TUFLOW FV


Test:5-Point:5
Velocity (ms^{-1})
Time (hours)
Flowroute
InfoWorks2D
ISIS2D
JFLOW-GPU
MIKE FLOOD
SOBEK
TUFLOW
UIM
ANUGA
TUFLOW FV


Test:5-Point:6
Water level (m)
Time (hours)
Flowroute
InfoWorks2D
ISIS2D
JFLOW-GPU
MIKE FLOOD
Dynamic RFSM
SOBEK
TUFLOW
UIM
Direct RFSM
ANUGA
TUFLOW FV


Test:5-Point:6
Velocity (ms^{-1})
Time (hours)
Flowroute
InfoWorks2D
ISIS2D
JFLOW-GPU
MIKE FLOOD
SOBEK
TUFLOW
UIM
ANUGA
TUFLOW FV



Full models

Arrival times and water levels

Predictions by the full models can be described in general terms as follows. Predictions of flood arrival times were consistent between most models within a maximum range of ~25min at point 5 (bottom of the valley). This is to be compared to a travel time of almost 3 hours since the start of the flood. Peak water levels were consistent with each other within ~0.4m at points 1,2,3,4,7, which at most represented ~10% of the predicted peak depth at these locations (which were all in excess of ~3m). However at Point 6, located close to Point 1 but at a higher elevation the discrepancies in peak levels were similar to those at Point 1, but represent a larger proportion of the predicted depths (equal to ~1.5m).

Exceptions to the above include the delayed arrival time (by about ~1hour) predicted by ANUGA at point 5 (following already existing delays, albeit smaller at upstream points, e.g. ~10mins at Point 4), and the slightly higher peak levels predicted by ISIS at some output points (at most ~0.6m at point 6).

At Point 5 (located in a ~2.5km^2 large pond at the downstream end of the valley where the water finally settles after filling any depressions located further upstream), final water levels were all within a 0.5m range, except those predicted by ANUGA (~0.5m below) and ISIS (~0.9m above). Differences there occur as the result of differences further upstream and are therefore larger. The discrepancy observed in the ISIS results is explained by a spurious volume gain in the solution, which may also explain the slightly higher peaks observed at other points[19].

Final water levels at other points were usually within a ~0.2m range (with some exceptions e.g. ANUGA at point 4). These are due to topography effects occurring because of the differences in the way models process DEM elevations. These topography effects may also explain partly some differences reported above, such as those in the arrival times (e.g. ANUGA at Point 5).

[19] As commented on by Halcrow: “Due to the way wetting and drying is handled, a discrepancy can arise between recorded volume in the domain and volume output in depth grids. This discrepancy rises to 40% of total volume toward the end of this simulation. It is likely to be due to the large areas drying in the simulation, leaving small negative depths at the drying cells (which are then filtered out when depth grids are written to disk). It may be possible to reduce the discrepancy through using a smaller depth threshold.”

Velocities

Peak velocities predicted by the different full models were within a relatively wider range than peak water levels, for example 1.6 to 2.2 m/s at Point 4, 1.9 to 3.1 m/s at Point 6, 1.1 to 1.6 m/s at Point 7. At other points (e.g. 2 and 3) larger differences are observed (with MIKE FLOOD predicting a larger peak value). However at these points the maximum velocities were too short-lived for meaningful peak values to be output considering the 1min time resolution of the output data. This suggests that a high output resolution (in time) is needed in dam break problems.

Simplified models

Arrival times and water levels:

Flowroute and JFLOW-GPU: predicted water levels and arrival times within the range of predictions by the full models at all output points (with some small differences e.g. slightly lower peaks at points 1 and 2).

UIM: UIM's water levels and arrival times are also generally within the range of predictions by the full models, although with delays in arrival times similar to those observed with ANUGA at the bottom of the valley, which may be partly explained by topography effects. However, later comments on UIM's prediction of velocities should be noted and may raise questions over the reliability of the water level and arrival time predictions.

RFSM Dynamic: predicted up to 100% longer travel times and peak water levels up to ~2m larger (as well as oscillatory solutions at several points).

RFSM Direct does not predict any transient peak levels or travel time, only final levels which have almost no practical relevance in a dam failure scenario. It is acknowledged that the final levels predicted in this test are in agreement with the full models' predictions.

Velocities:

The velocities predicted by the simplified models (except the RFSM for which no velocity outputs were provided) were partly consistent with those predicted by the full models, however:

Flowroute: predicts significantly higher peaks at Points 3 and 5 and a strongly oscillatory solution at points 4 and 5.

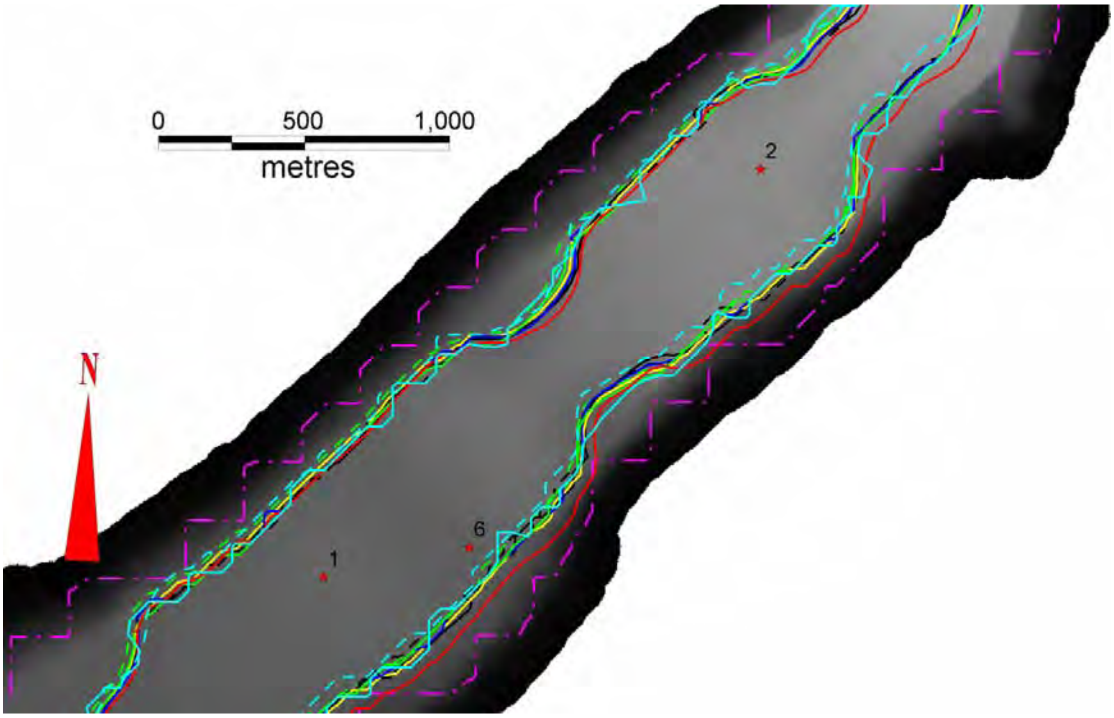
JFLOW-GPU: produces a strongly oscillatory solution at Points 4 and 5 during the final phase of the flood and slightly different peak values, e.g. Point 7.

UIM: predicts higher peaks at Points 5 and 7 and oscillatory patterns at Points 2 and 6 during the receding phase of the flood.

4.5.3 Output in raster format

Note: the GIS software available to Heriot Watt University was unable to import the ANUGA peak depth and level grids provided[20].

Figure 9: 0.5m contour lines of peak depths for an section of floodplain around Points 1,2,and 6. Colour coding as in the rest of the report.



[20] Despite these being in a compatible format.

Figure 10: 3m/s contour lines of peak velocities for the same area of floodplain as in previous figure.

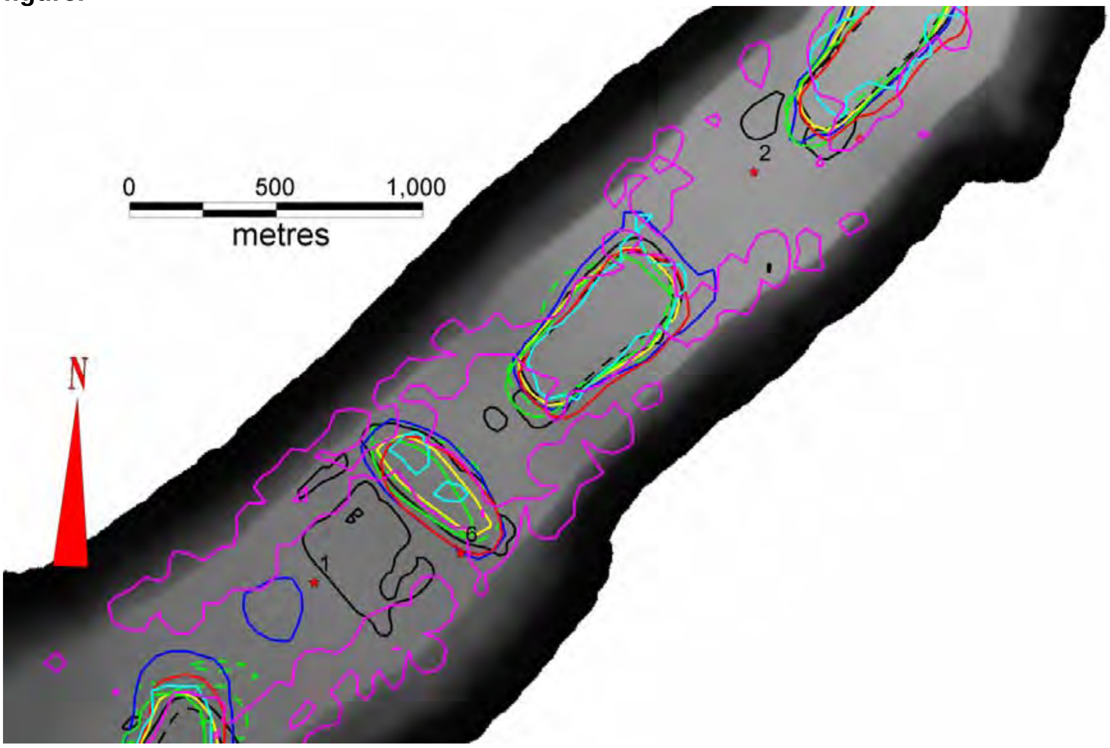
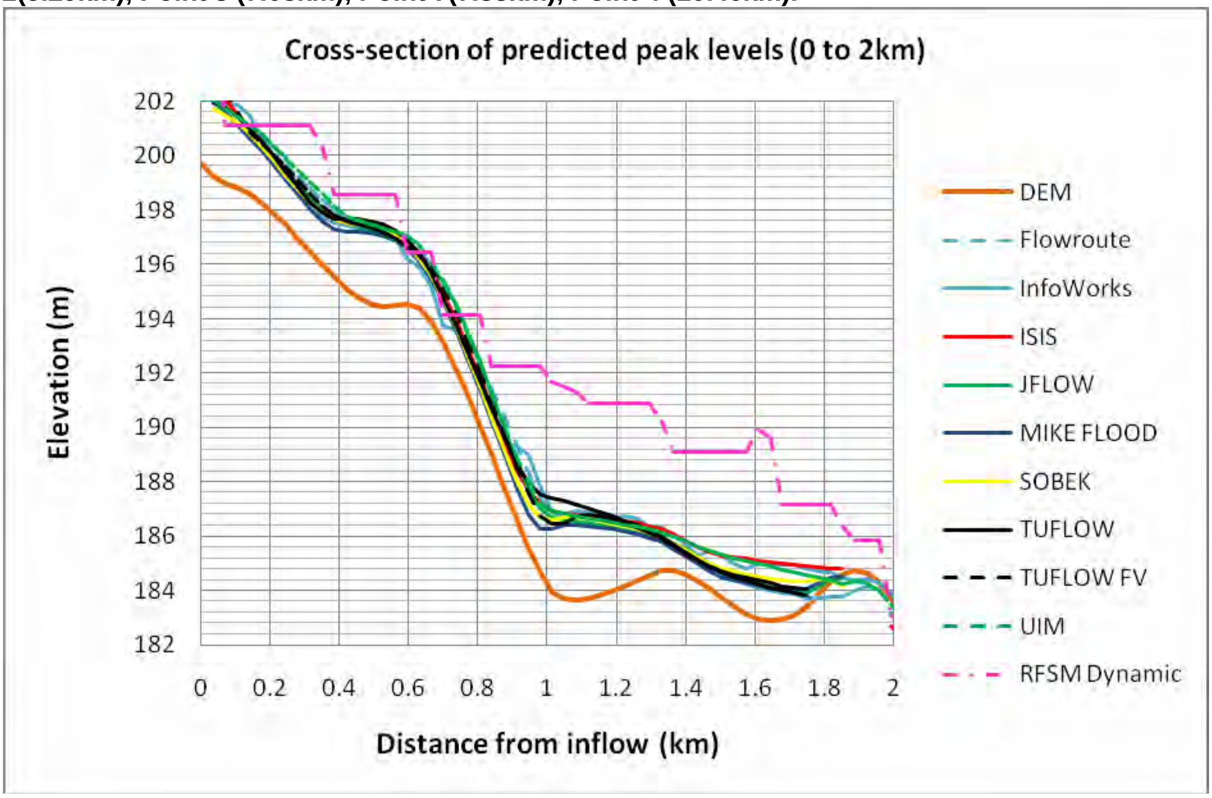
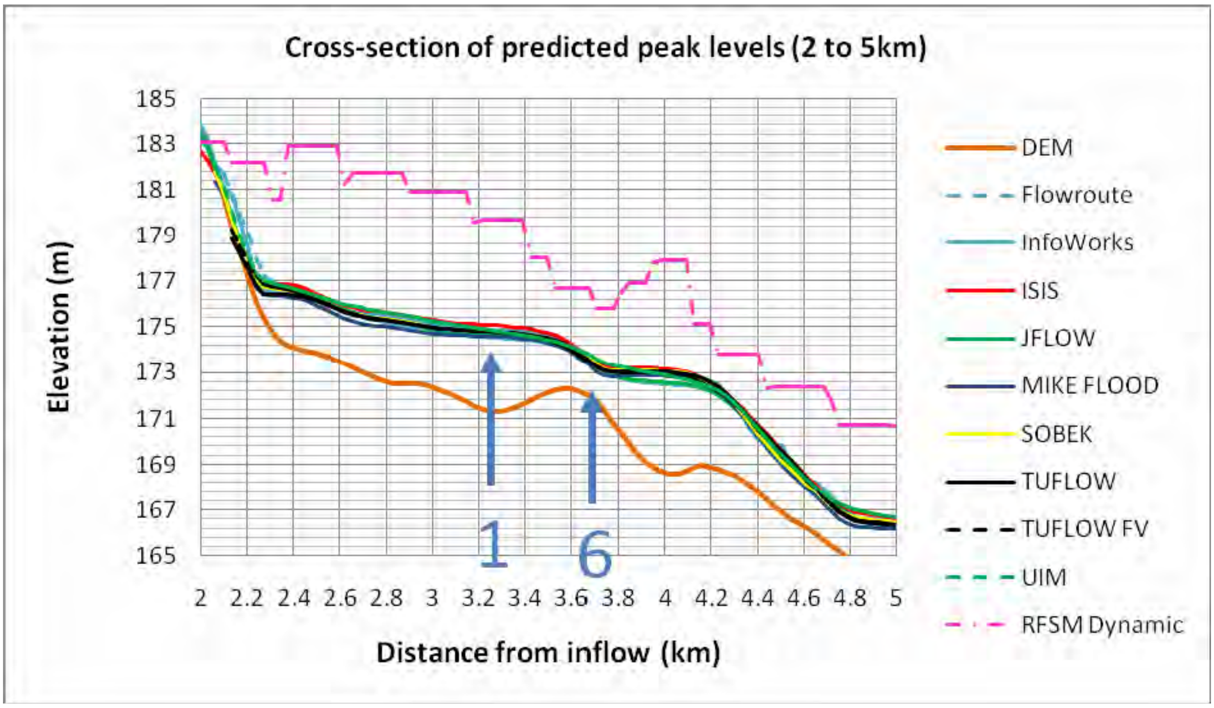
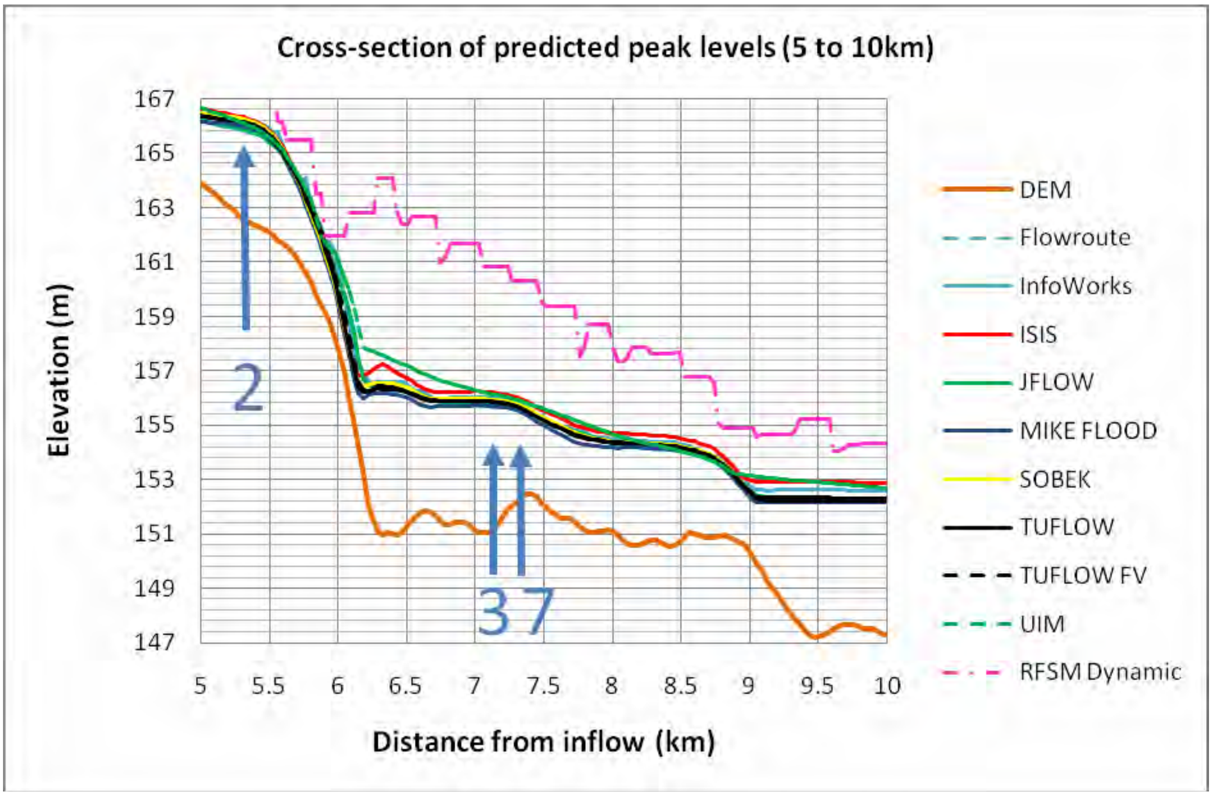


Figure 11: Cross-section of peak levels along the valley centre line, see Figure 8, with approximate locations of time series output points: Point 1 (3.24km), Point 6 (3.67km), Point 2(5.29km), Point 3 (7.08km), Point 7(7.33km), Point 4 (10.46km).




Cross-section of predicted peak levels (2 to 5km)
Elevation (m)
185
183
181
179
177
175
173
171
169
167
165
2
2.2
2.4
2.6
2.8
3
3.2
3.4
3.6
3.8
4
4.2
4.4
4.6
4.8
5
Distance from inflow (km)
1
6
DEM
Flowroute
InfoWorks
ISIS
JFLOW
MIKE FLOOD
SOBEK
TUFLOW
TUFLOW FV
UIM
RFSM Dynamic


Cross-section of predicted peak levels (5 to 10km)
Elevation (m)
167
165
163
161
159
157
155
153
151
149
147
5
5.5
6
6.5
7
7.5
8
8.5
9
9.5
10
Distance from inflow (km)
2
3
7
DEM
Flowroute
InfoWorks
ISIS
JFLOW
MIKE FLOOD
SOBEK
TUFLOW
TUFLOW FV
UIM
RFSM Dynamic

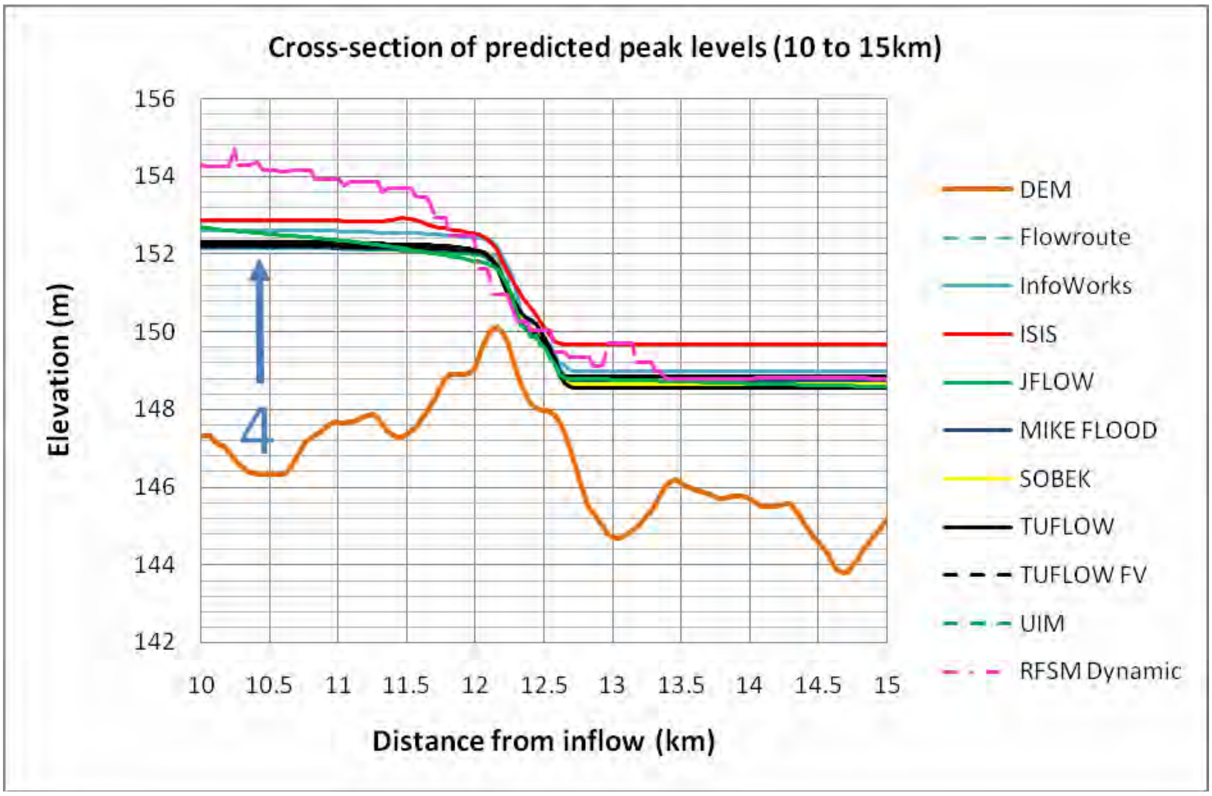
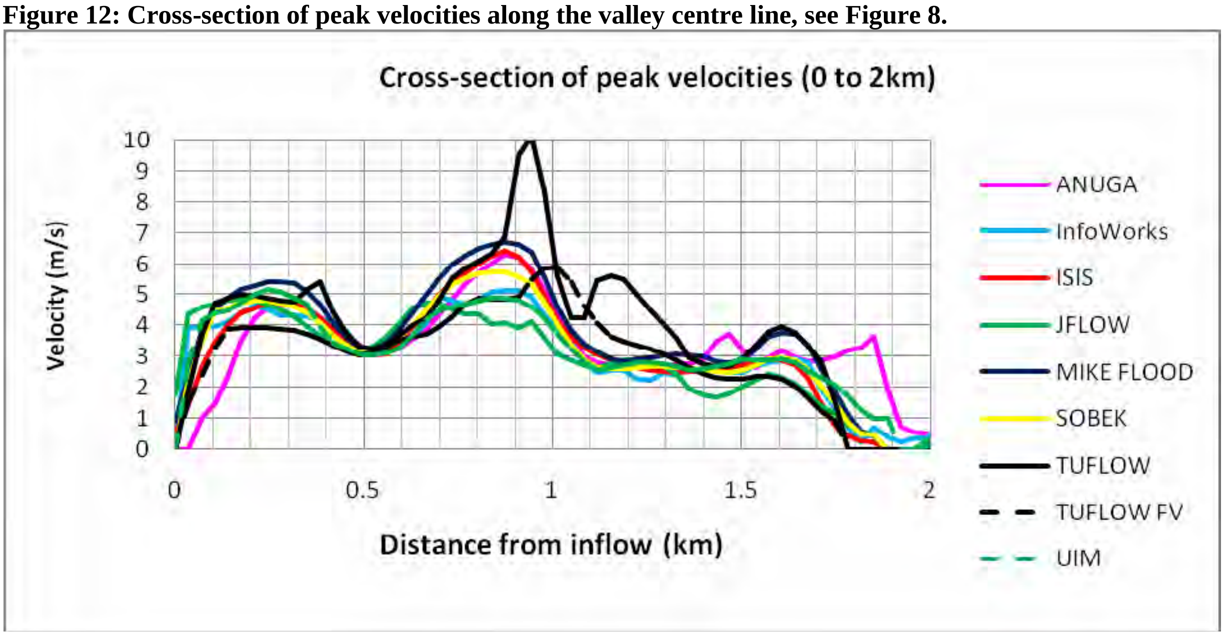
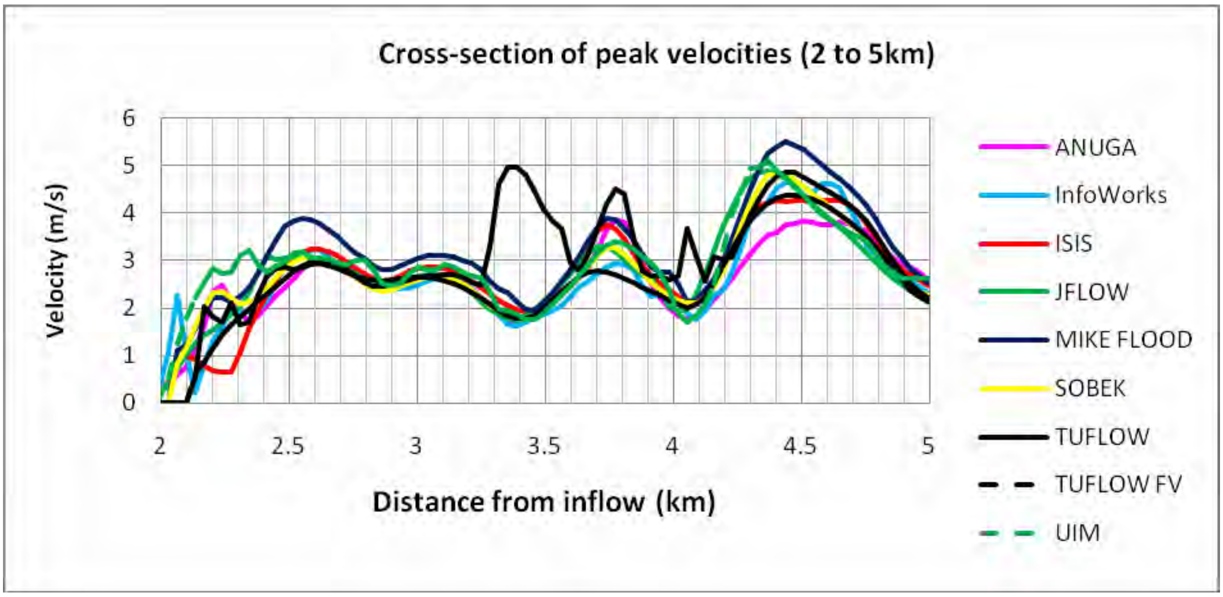
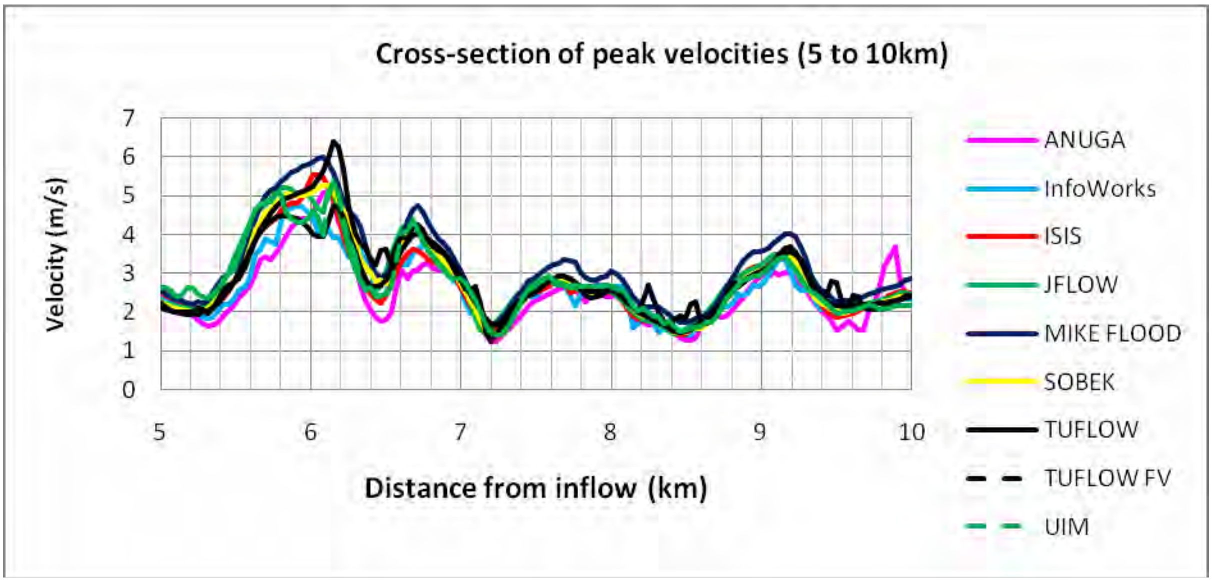
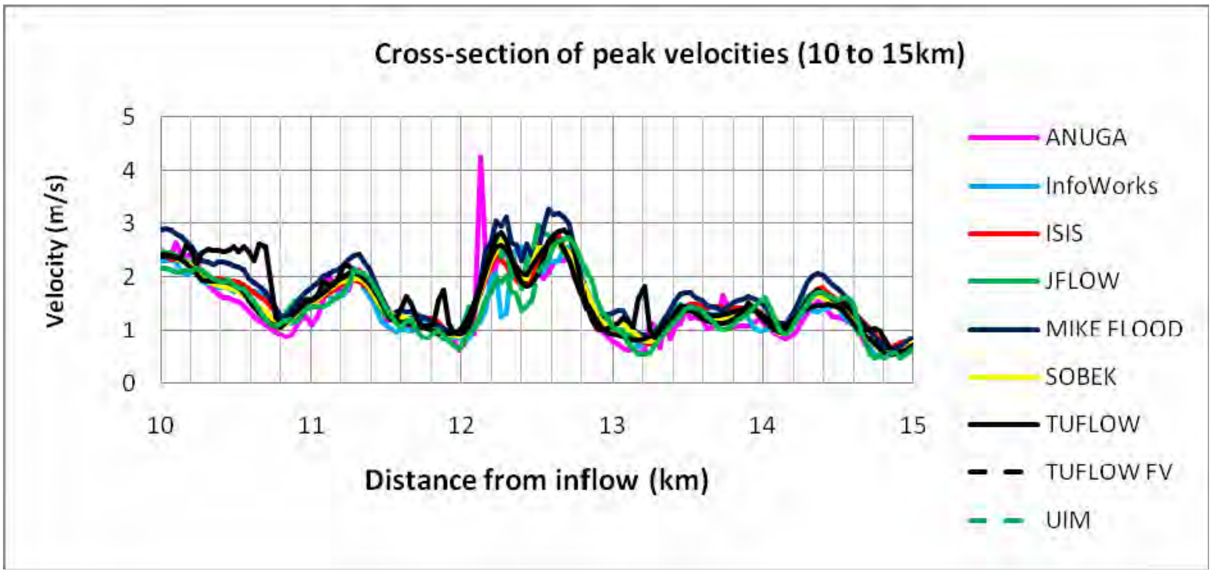


Figure 12: Cross-section of peak velocities along the valley centre line, see Figure 8.




Cross-section of peak velocities (2 to 5km)
Velocity (m/s)
0
1
2
3
4
5
6
2
2.5
3
3.5
4
4.5
5
Distance from inflow (km)
ANUGA
InfoWorks
ISIS
JFLOW
MIKE FLOOD
SOBEK
TUFLOW
TUFLOW FV
UIM


Cross-section of peak velocities (5 to 10km)
Velocity (m/s)
0
1
2
3
4
5
6
7
5
6
7
8
9
10
Distance from inflow (km)
ANUGA
InfoWorks
ISIS
JFLOW
MIKE FLOOD
SOBEK
TUFLOW
TUFLOW FV
UIM


Cross-section of peak velocities (10 to 15km)
Velocity (m/s)
0
1
2
3
4
5
10
11
12
13
14
15
Distance from inflow (km)
ANUGA
InfoWorks
ISIS
JFLOW
MIKE FLOOD
SOBEK
TUFLOW
TUFLOW FV
UIM

Observations from Figure 9 to Figure 12 are broadly consistent with those from the time series. In addition, Figure 9 suggests discrepancies in the inundation outlines predicted by most models, due to differences in level predictions and to topography effects (for example ISIS predicts the outline to be ~50m to ~100m further to the south than most models). RFSM Dynamic predicts outlines up to ~250m away from the others, due to its oscillatory behaviour returning very high peak levels. This latter result is also shown by the cross-section of peak levels (Figure 11).

Figure 10 suggests that larger areas of high velocities were predicted by TUFLOW and MIKE FLOOD, which is consistent with the presence of sharp peaks observed at some output points in the time series. This is also shown by the cross-section of peak velocities (Figure 12), particularly in the case of TUFLOW.

It can also be observed that large areas of high velocity were predicted by ANUGA along the edges of the valley (shallow flow). Some high peaks are also visible in the cross-section (Figure 12).This is likely to be due to spurious oscillations in the solution.

4.5.4 Conclusions from Test 5

All **full models** predicted similar results in terms of travel times (within ~20%), peak water levels (within 0.4m). These differences are unlikely to be larger than the required accuracy of predictions for a problem of this type and scale in a practical application. The differences observed in the peak water levels however suggest that predictions of inundation extent along the edges of inundation where depths are shallow are unlikely to be consistent between packages[21].

The differences observed in the velocity predictions by the full models (up to ~30%) suggest that predictions of hazard by any particular model are likely to be only indicative.

In addition, the lack of robustness observed in the predictions by the models without shock-capturing capabilities suggest that these are likely to be important. ANUGA also suffered numerical oscillations in this test.

The comments above are more likely to be pertinent in practical applications based on high resolution DEMs (e.g. 1m LiDAR) where topographic effects will be more significant (the DEM used in Test 5 is an artificially smoothed 10m resolution DEM).

The **simplified models** Flowroute, JFLOW-GPU and UIM made predictions of water levels and arrival times consistent with those of the full models, differences in magnitude are unlikely to exceed those required for modelling accuracy in practical applications with this type of flood scenario. The same conclusions as above (full models) therefore apply, with the same reservations, to Flowroute, JFLOW-GPU and UIM, as far as levels and timings are concerned. These three models made significantly less comparable predictions of velocities.

The RFSM Dynamic predicted significantly different arrival times and peak levels (with oscillatory solutions indicating a lack of robustness in the numerical solution), with differences in predictions that would be significant in flood risk management applications of this type. Velocity predictions by RFSM are at the present time not available.

The RFSM Direct is inadequate for this type of application.

No conclusions can be drawn at this stage concerning Flood Risk Mapper as the test was not undertaken due to lack of resources.

[21] Even if a very high resolution DEM is used.

4.5.5 Summary of relevant technical information

TEST 5 (1) Name	(2) Version (+ numeric. scheme)	(3) Hardware	(4) Multi-Proc.	(5) Grid (50m or 7600 elements)	(6) Time-stepping	(7) Run time (min)
ANUGA	1.1beta_7501	Intel Mobile Core 2 Duo T7500 (Merom) 2.2 GHz RAM 2048 MB (DDR2)	no	7828 elements	adaptive	69.3
Flood Risk Mapper	*Not tested*					
FloodFlow	W.12.0 Beta ADI	Intel(R) Core™ 2 CPU 6400 2.13GHz RAM 2GB	no	50m	Adaptive	350
Flowroute	2.9.8	2.4Ghz (Intel Q6600) RAM 4GB	OMP	50m	0.02s	112
InfoWorks	RS 2D v10.5	Intel® Core™ i7 920 2.66GHz Quad Core 12 GB RAM (DDR3-1333)	OMP	7648 triangles	Adaptive	0.7
ISIS	3.2.0.21 TVD	Intel Core 2-Quad CPU Q6600 2.4 GHz RAM 2.0 GB	no	50m	1s	47.0
JFLOW-GPU	JFLOW-GPU DW	AMD Phenom II X4 940 3.0 GHz RAM 2.25 GB GPU: NVIDIA GeForce GTX 295	Yes - GPU	50m	Avg 0.1217s	10.2
MIKE FLOOD	2009 incl. Serv. Pack 2	Intel Core 2 Duo CPU P8600 2.4 GHz RAM 4.00 GB	no	50m	10s	0.68
RFSM (Direct)	*3.5.4*	Intel Dual Xeon 2 cores of 3GHz RAM 2GB	no	58 ele-ments[22]	N/A	<1s
RFSM (Dynamic)	0.1 (Beta)	Intel Dual Xeon 2 cores of 3GHz RAM 2GB	no	616 ele-ments[23]	30s	9.8
SOBEK	2.13	Intel i7 8 core CPU 2.66 GHz	no	50m	10s	2.8
TUFLOW	2010-01-AD-iSP	Intel Core 2 Duo T9800 2.93GHz RAM 4Gb	no	50m	20s	0.6
TUFLOW FV	0.107.0006 2nd order solution	Intel Xeon X5472 3.00GHz RAM 8Gb	Yes – 8 CPUs	7424 nodes	Typical time step: 0.7 - 1s	2.9 (1.4)
UIM	2009.12	Dual Quad-core 2.83GHz Intel Xeon E5440 Harpertown node RAM 16GB	OMP	50m	0.5s	44.5

[22] See Appendix B.6
[23] See Appendix B.6

Other information provided:

Flowroute: “Output of a maximum velocity grid is not included within the version of Flowroute used for this exercise. This feature will be included within future releases.”

RFSM (Dynamic): “Total simulation time vary significantly with the frequency of outputs writing. This is due to the fact that model data and results are stored in a SQL database, to be compatible with the NaFRA and MDSF2 data structure. The RFSM run times would be shorter if the model were to write results on the local hard drive.”

Final volumes on floodplain, as a % variation compared to the expected volume, equal to the cumulative inflow, ie. 9450000 m^3 (assuming the use of linear interpolation at the boundary, as specified).

Flowroute	0%
InfoWorks	+ 0.045%
ISIS	+ 3.809% (however see footnote number 19)
JFLOW-GPU	-0.022%
MIKE FLOOD	+1.654%
SOBEK	0%
TUFLOW	-0.60%
TUFLOW FV	-0.003%
UIM	0%

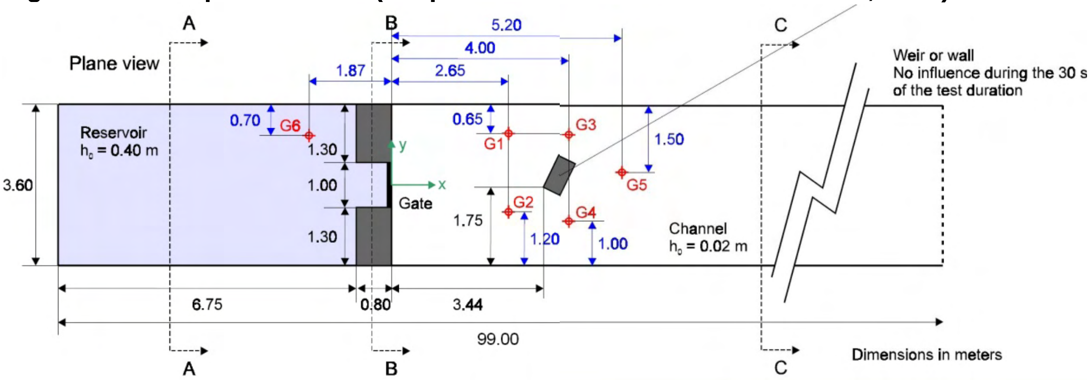
4.6 Test 6: Dam break

4.6.1 Test 6A: laboratory scale

4.6.1.1 Introduction

This dam-break test case (see Appendix A.6 for details) is the original benchmark test case available from the IMPACT project (Soares-Frazao and Zech, 2002), for which measurements from a physical model at the Civil Engineering Laboratory of the Université Catholique de Louvain (UCL) are available. The physical dimensions are those of the laboratory model. The test involves a simple topography, a dam with a 1m wide opening, and a building downstream of the dam, see Figure 13. An initial condition is applied, with a uniform water level of 0.4m upstream from the dam, and 0.02m downstream.

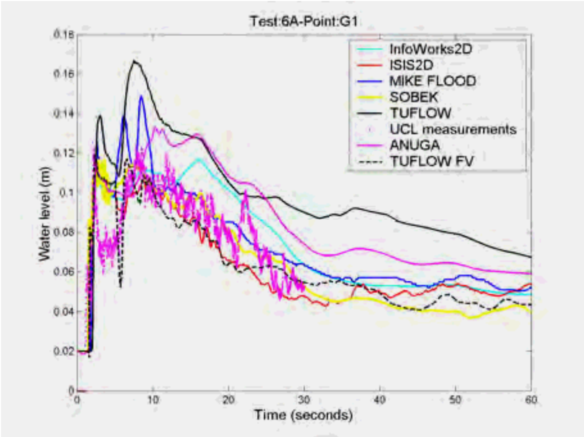
Figure 13: Set-up for Test 6A (adapted from Soares-Frazao and Zech, 2002).

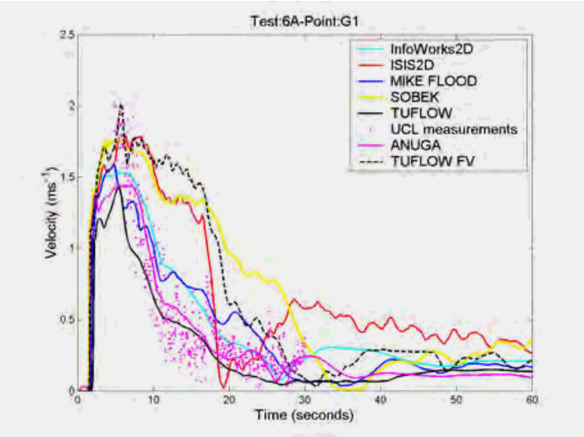


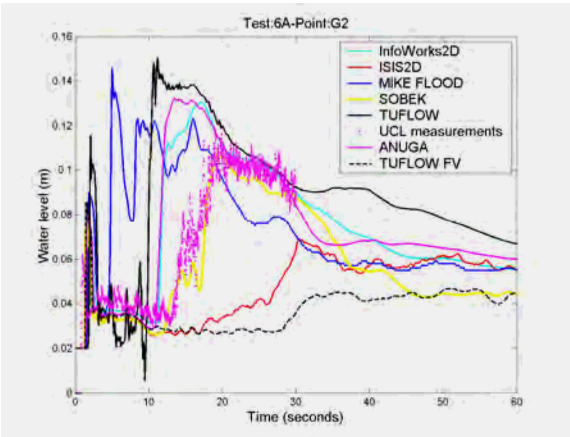
Measured water levels and velocities were provided by UCL for 6 points (G1 to G6, except velocities at G6). Some of the measurement data are missing for part of the time, particularly velocities.

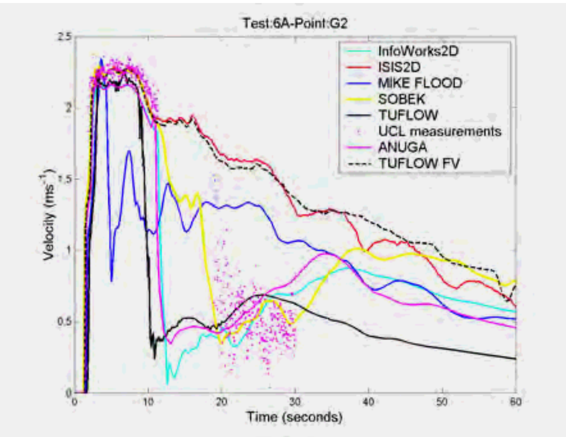
The objective of the test is to assess the package's ability to simulate hydraulic jumps and wake zones behind buildings using high-resolution modelling.

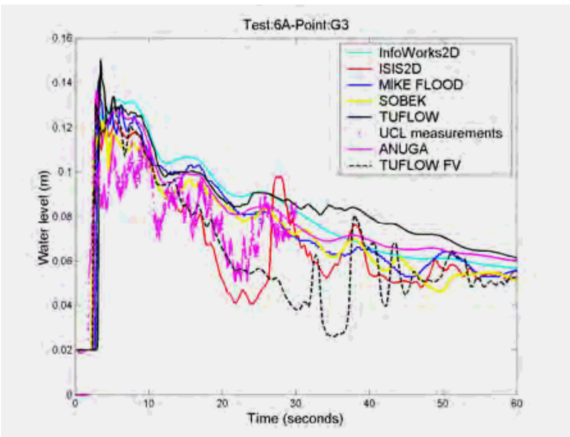
4.6.1.2 Water level and velocity time series

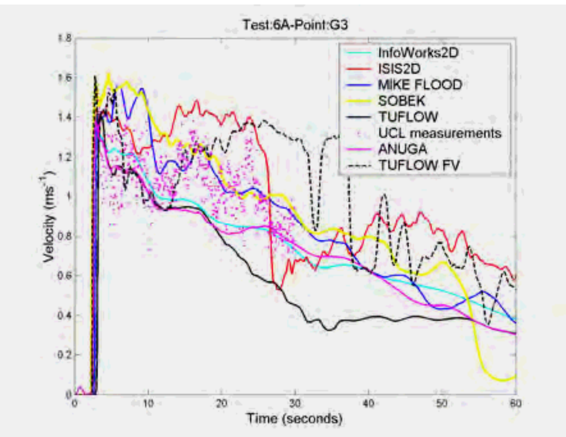

Test:6A-Point:G1
InfoWorks2D
ISIS2D
MIKE FLOOD
SOBEK
TUFLOW
UCL measurements
ANUGA
TUFLOW FV
Water level (m)
Time (seconds)

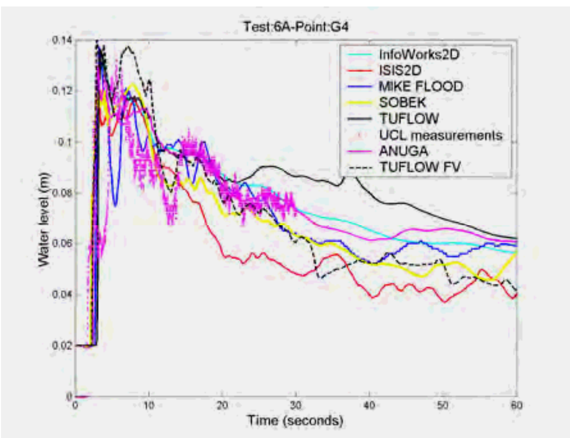

Test:6A-Point:G1
InfoWorks2D
ISIS2D
MIKE FLOOD
SOBEK
TUFLOW
UCL measurements
ANUGA
TUFLOW FV
Velocity (ms-1)
Time (seconds)

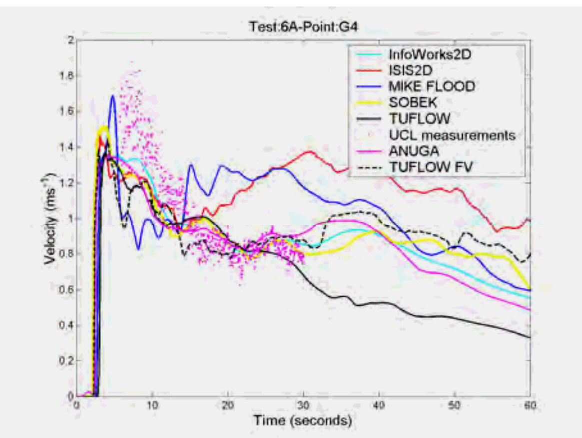

Test:6A-Point:G2
InfoWorks2D
ISIS2D
MIKE FLOOD
SOBEK
TUFLOW
UCL measurements
ANUGA
TUFLOW FV
Water level (m)
Time (seconds)

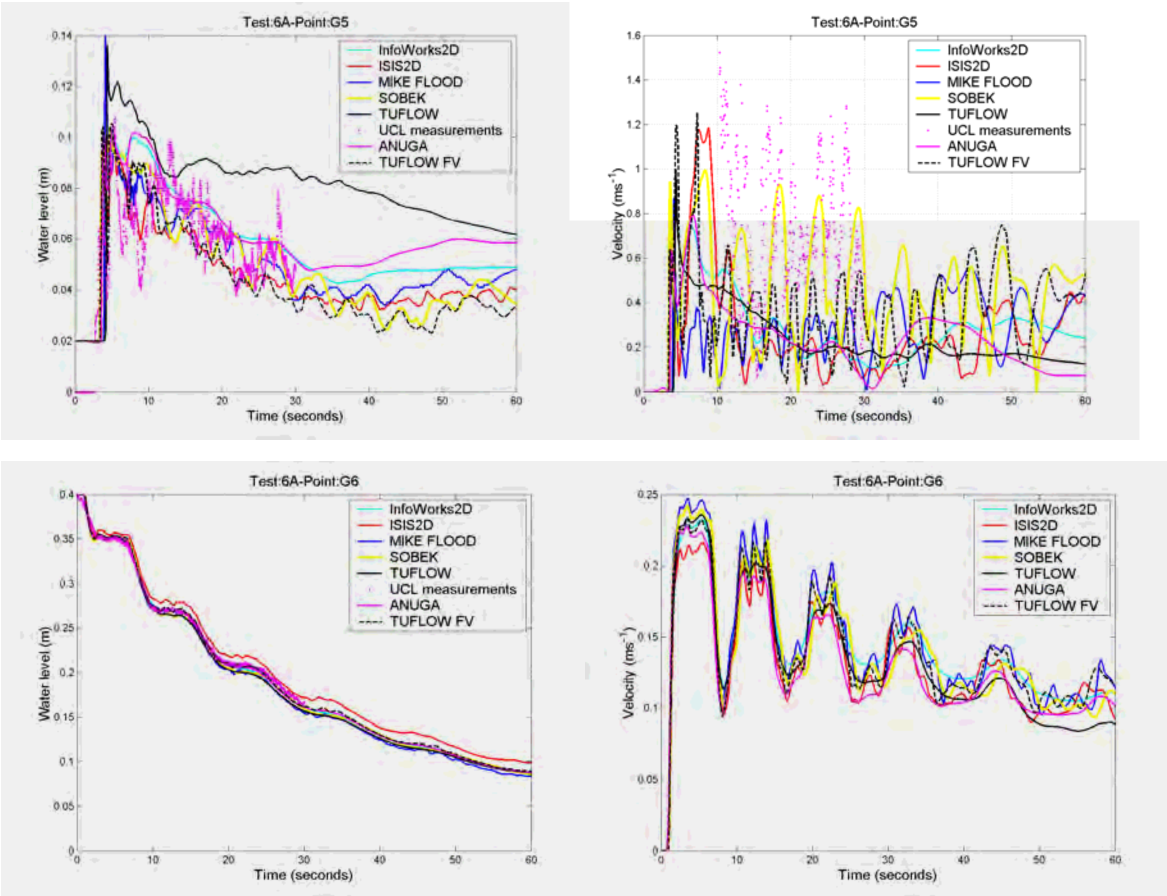

Test:6A-Point:G2
InfoWorks2D
ISIS2D
MIKE FLOOD
SOBEK
TUFLOW
UCL measurements
ANUGA
TUFLOW FV
Velocity (ms-1)
Time (seconds)


Test:6A-Point:G3
InfoWorks2D
ISIS2D
MIKE FLOOD
SOBEK
TUFLOW
UCL measurements
ANUGA
TUFLOW FV
Water level (m)
Time (seconds)


Test:6A-Point:G3
InfoWorks2D
ISIS2D
MIKE FLOOD
SOBEK
TUFLOW
UCL measurements
ANUGA
TUFLOW FV
Velocity (ms-1)
Time (seconds)


Test:6A-Point:G4
InfoWorks2D
ISIS2D
MIKE FLOOD
SOBEK
TUFLOW
UCL measurements
ANUGA
TUFLOW FV
Water level (m)
Time (seconds)


Test:6A-Point:G4
InfoWorks2D
ISIS2D
MIKE FLOOD
SOBEK
TUFLOW
UCL measurements
ANUGA
TUFLOW FV
Velocity (ms-1)
Time (seconds)



It can be observed first that the UCL measurements exhibit high frequency oscillations of amplitude typically ~0.01m (water levels) and ~0.2 ms^{-1} or more (velocities). These oscillations may be measurement errors or physical oscillations due to chaotic turbulent patterns in the flow. None of the model predictions replicated this, partly due to the insufficient space resolution (0.1m) or time resolution of the output (0.1s), and partly to the inherent inability of shallow water equation based models to replicate turbulent processes, significant free surface deformation and vertical accelerations. Comments from the above graphs can be made as follows, concerning the initial phase (30s) for which UCL measurements were available.

At point G6 upstream from the gate water level predictions were in excellent agreement with UCL measurements (no velocity measurements were available at G6), reflecting the periodical behaviour of the flow through the gate with a period ~9s. Discrepancies in depth predictions at G6 had a relative magnitude of ~5% at most, suggesting that the flow through the gate was accurately modelled. Any consistent observations at all other points cannot be made. Most models often predicted water level variations within the range in which the UCL measurements oscillated, except, most notably, at G2 and in the first few seconds at G1,G3,G4. Comments point by point can be made as follows:

G1: Good predictions between t~6s and t=30s by all models except TUFLOW and ANUGA (predicted values were too high by up to ~50%). However TUFLOW and ANUGA made the best predictions of velocities during the same time period, which were overestimated by TUFLOW FV, SOBEK and ISIS 2D (by up to ~150%). The best predictions overall (levels *and* velocities) are those of MIKE FLOOD and InfoWorks.

G2: All models made a correct prediction of the initial supercritical flow (both in level and velocity) although the timing of the hydraulic jump (sudden rise in depth and drop in velocity) was predicted too early by MIKE FLOOD, too early to a lesser extent by TUFLOW (which also produced an oscillatory solution), ANUGA and InfoWorks, and too late (if at all) by TUFLOW FV and ISIS.The prediction by SOBEK was best.

G3, G4: All models captured the sharp initial peak in depth and velocity, although with an overestimation of up to ~15%, and some of the subsequent decrease, although failed to predict some short lived patterns in the measured levels (such as the sudden drop in depth at G4 at t~4s). TUFLOW and MIKE FLOOD predicted a sharp peak at G5, higher than the measured values. TUFLOW levels at G5 were too high for most of the time (by up to ~50%). Most models predicted oscillatory velocity patterns at point G5, also observed in the measurements, albeit with different amplitudes and periods.

While some accurate predictions can be noted (e.g. SOBEK at G2) these are not necessarily corroborated at all points for both depth and velocity. It can generally be observed that the level of agreement was generally poorer for velocities than for depths. The *range* of velocities (irrespective of timing) was more accurately modelled. The observations above may be explained by the inability of the models to replicate measurements in this highly variable (chaotic) flow, and by inherent limitations in modelling using the shallow water equations.

Any superiority (in accuracy) of the “shock-capturing” schemes over the non shock-capturing schemes[24] is only demonstrated in that the latter produce sharp initial peaks (likely to be unphysical, e.g. at G2, G3, G5), with therefore a tendency to overestimate peak values (which are important to applications in flood management).

Note: DHI also supplied results from a MIKE Flexible Mesh simulation (the Finite Volume solver within MIKE FLOOD, with shock-capturing capability), not represented here. These were similar to the results by other shock-capturing models (with InfoWorks being the closest).

4.6.1.3 Conclusions form Test 6A

Test 6A did not demonstrate conclusively any superior ability of any of the **full models** to accurately predict hydraulic jumps and wake zones around buildings in a consistent manner at the scale of physical model data. Ranges of variability, i.e. peaks (which are important in flood management applications) were predicted rather than values of water levels at specific times. Shock capturing schemes tended however to perform better in this respect than TUFLOW (FD), MIKE FLOOD (FD).

It was also confirmed by the developers of the **simplified models** (Flood Risk Mapper, Flowroute, RFSM, UIM) that these are unlikely to produce realistic results in an application similar to test 6A.

[24] It must however be pointed out that the results by TUFLOW (FD only) were somewhat improved between the draft version and the final version of this report through calibration of the eddy viscosity coefficient. All other results were obtained without knowledge of the UCL measurements.

4.6.1.4 Summary of relevant technical information.

Note: 'N/A' in column (8) refers to the fact that the model does not account for eddy viscosity.

TEST 6A (1) Name	(2) Version (+ numeric. scheme)	(3) Hardware	(4) Multi-Proc.	(5) Grid (0.1m or 36000 elements)	(6) Time-stepping	(7) Run time (min)	(8) Eddy Visc. (m^2/s)
ANUGA	1.1beta_7501	Intel Mobile Core 2 Duo T7500 (Merom) 2.2 GHz RAM 2048 MB (DDR2)	no	37046 elements	adaptive	11.5	N/A
Flood Risk Mapper	*Not tested*						
FloodFlow	*Not tested*						
Flowroute	*Not tested*						
InfoWorks	RS 2D v10.5	Intel® Core™ i7 920 2.66GHz Quad Core 12 GB RAM (DDR3-1333)	OMP	37147 triangles	Adaptive	1.3	N /A
ISIS	3.3 TVD	Intel Core 2-Quad CPU Q6600 2.4 GHz RAM 2.0 GB	no	0.1m	0.002s	362.0	0
JFLOW-GPU	*Not tested*						
MIKE FLOOD	2009 incl. Serv. Pack 2	Intel Core 2 Duo CPU P8600 2.4 GHz RAM 4.00 GB	no	0.1m	0.02s	1.29	0.004
RFSM (Direct)	*Not tested*						
RFSM (Dynamic)	*Not tested*						
SOBEK	2.13	Intel i7 8 core CPU 2.66 GHz	no	0.1m	0.02s	6.5	0
TUFLOW	2010-01-AD-iSP	Intel Core 2 Duo T9800 2.93GHz RAM 4Gb	no	0.1m	0.05s	2.6	0.05 S + 0.05 C[25]
TUFLOW FV	0.107.0006 2nd order solution	Intel Xeon X5472 3.00GHz RAM 8Gb	Yes – 8 CPUs	31687 nodes	Typical time step: 0.02s	7.1 (1.3)	0.2 S[26]
UIM	*Not tested*						

Other information provided:

ISIS: Simulations were also performed using the ADI scheme. Results were not stable or physically realistic.

[25] 0.05 (Smagorinsky) + 0.05 (Constant)
[26] 0.2 (Smagorinsky)

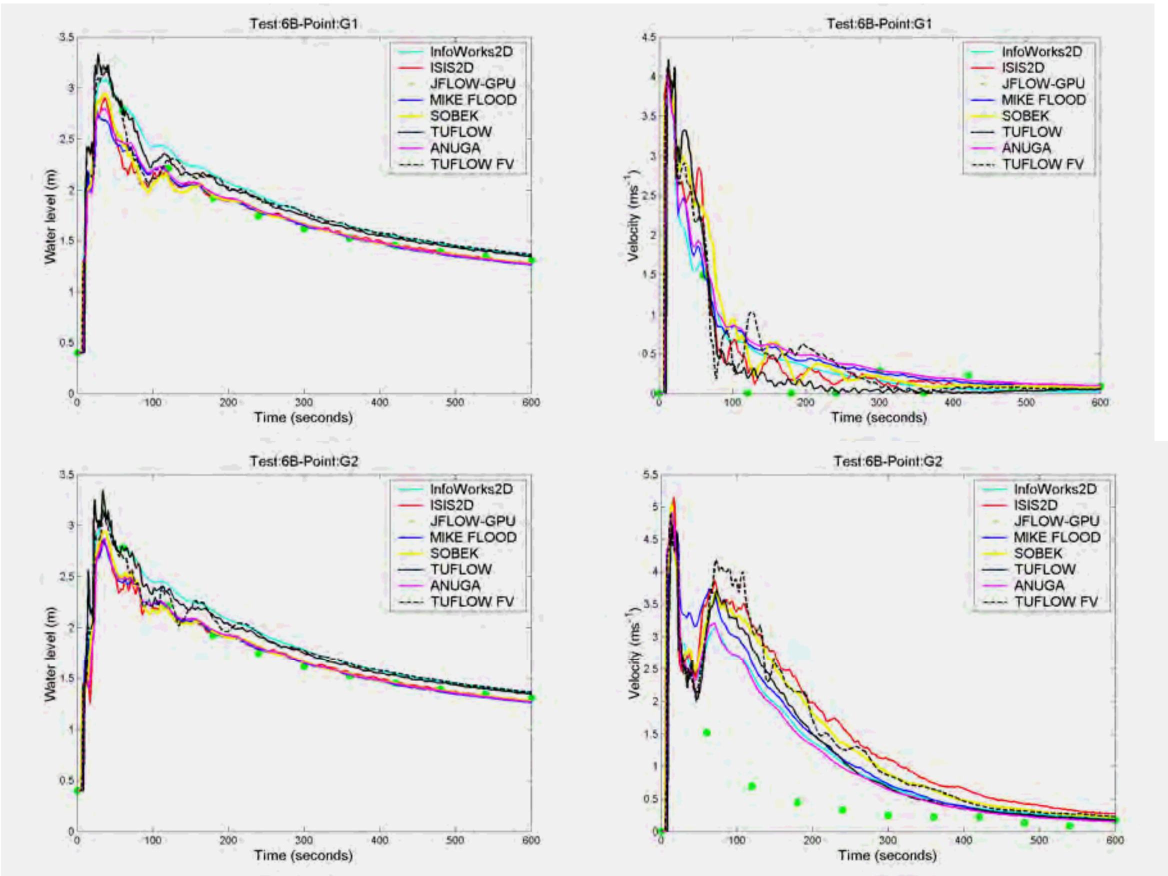
4.6.2 Test 6B: field scale

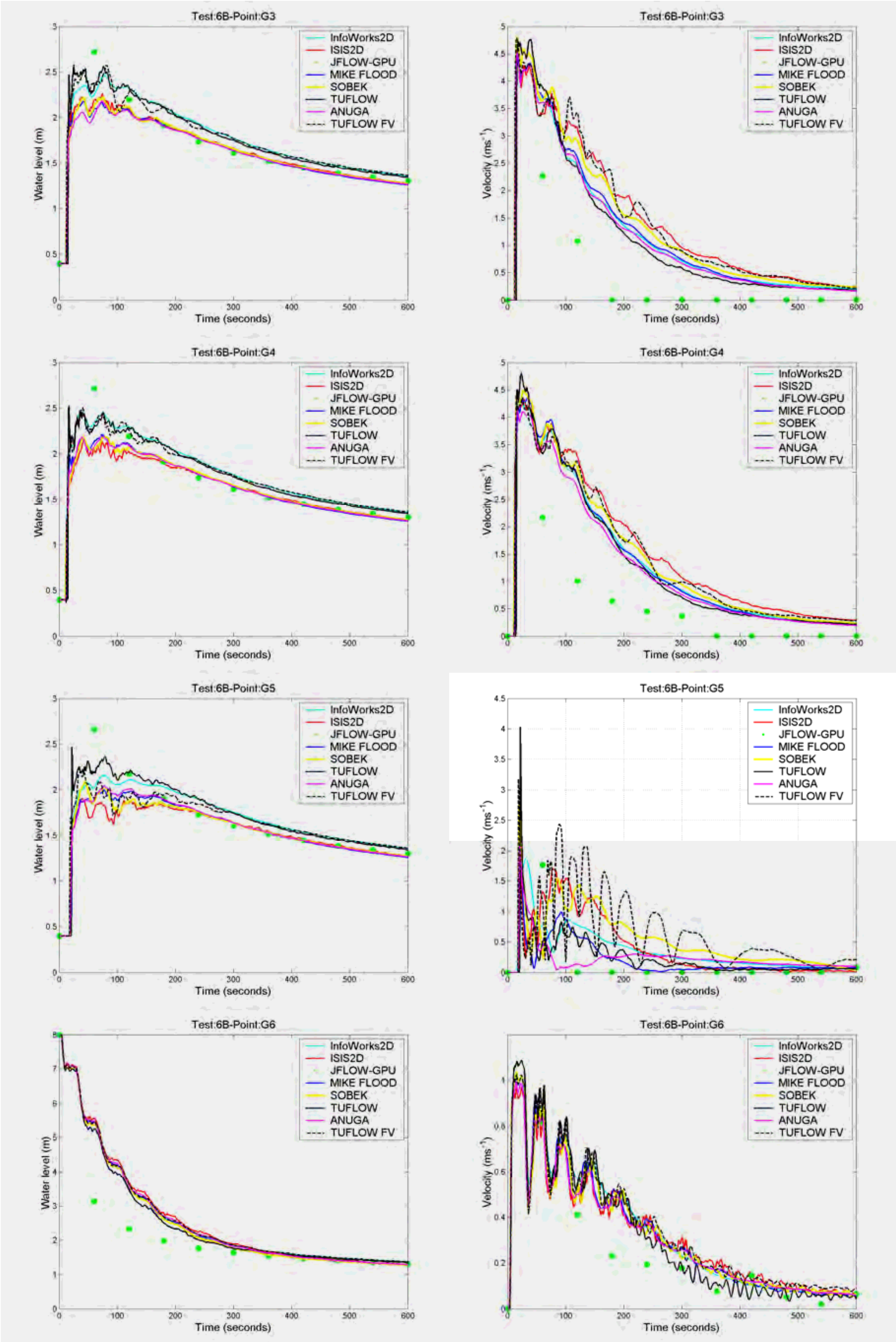
4.6.2.1 Introduction

This dam-break test case (see Appendix A.6 for details) has been adapted from the original IMPACT test case (Test 6A), where all physical dimensions (including the initial water levels) have been multiplied by 20 to reflect realistic dimensions encountered in flood inundation modelling applications. Thus the canal is 72m wide, the building 16m by 8m, and the initial condition consists of a uniform water level of 8m upstream from the dam, and 0.4m downstream.

The objective of the test is to assess the package's ability to simulate hydraulic jumps and wake zones behind buildings at the field scale using high-resolution (2m) modelling.

4.6.2.2 Water level and velocity time series





Test:6B-Point:G3
Test:6B-Point:G4
Test:6B-Point:G5
Test:6B-Point:G6
Water level (m)
Velocity (ms-1)
Time (seconds)
InfoWorks2D
ISIS2D
JFLOW-GPU
MIKE FLOOD
SOBEK
TUFLOW
ANUGA
TUFLOW FV

Full models:

All full models predicted a similar decrease in water level (within a 5 to 10% range) upstream from the constriction (point G6) after the removal of the dam, with a periodical behaviour similar to that observed in 6A.

At points downstream from the gate, there was a consistent agreement between many of the full model predictions, with predictions of peak depths being within a 15-20% of each other, or within a 30-40cm wide range. A similar comment can be made regarding peak velocities, with an even smaller range (up to ~10%), except at Point 5 were peak velocities were smaller and a wider range of behaviour was predicted. TUFLOW, TUFLOW FV and InfoWorks consistently predicted levels ~10% higher than SOBEK, ISIS, ANUGA and MIKE at G1, G2, G3, G4. This is unexplained[27].

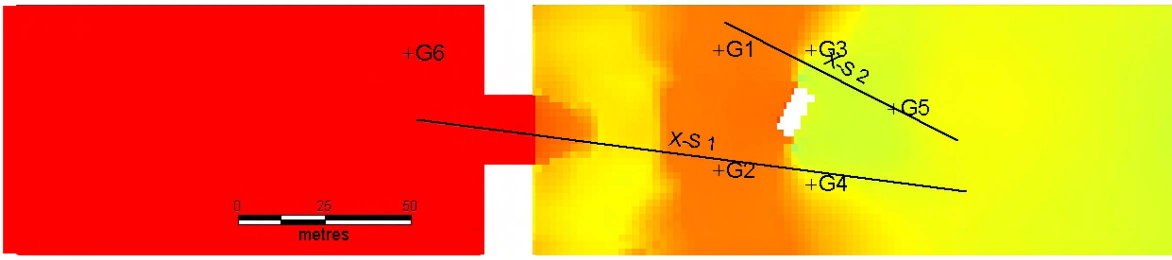
Simplified model (JFLOW-GPU):

JFLOW-GPU: A ~2 times faster decrease in water level compared with the full models is predicted at point G6, upstream from the gate, due to the simplified formulation used in JFLOW (which did not predict accurately the sudden collapse of water driven primarily by potential energy). This limits any opportunity to comment on the results obtained downstream from the gate. Also predictions of initial peak values are not available due to the output resolution of 1min. However subsequent level predictions agree in order of magnitude with the full models. The few velocity values predicted downstream from the gate are mostly lower than other model predictions, which would appear not to be consistent with the faster emptying of the reservoir.

4.6.2.3 Output in gridded format

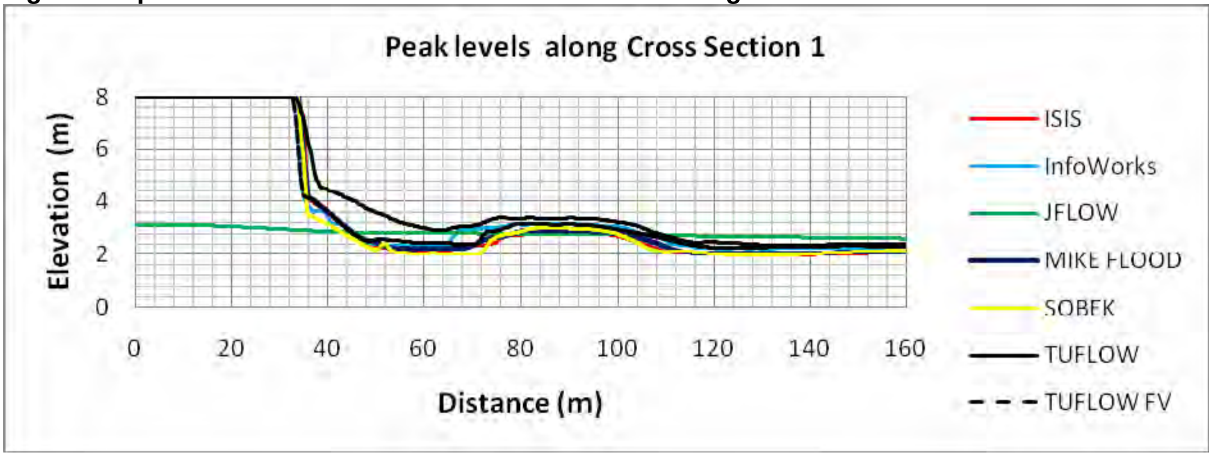
Note: The GIS software available to Heriot Watt University was unable to import the ANUGA grids provided[28].

Figure 14: plan view showing the hydraulic jump and the locations of the cross-sections (from a peak water level grid)



[27] I.e. does not appear to be explained by any discrepancies in the rate at which the flow through the gate happened, or by any differences in the shock-capturing capabilities of the models.
[28] Despite these being in a compatible format.

Figure 15: peak water level elevations and velocities along cross-section 1



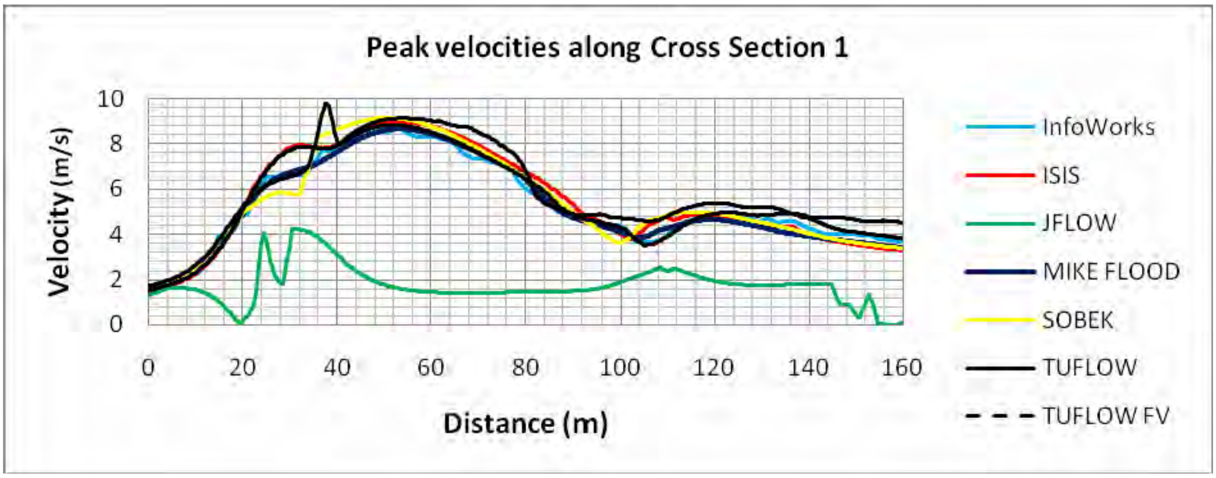
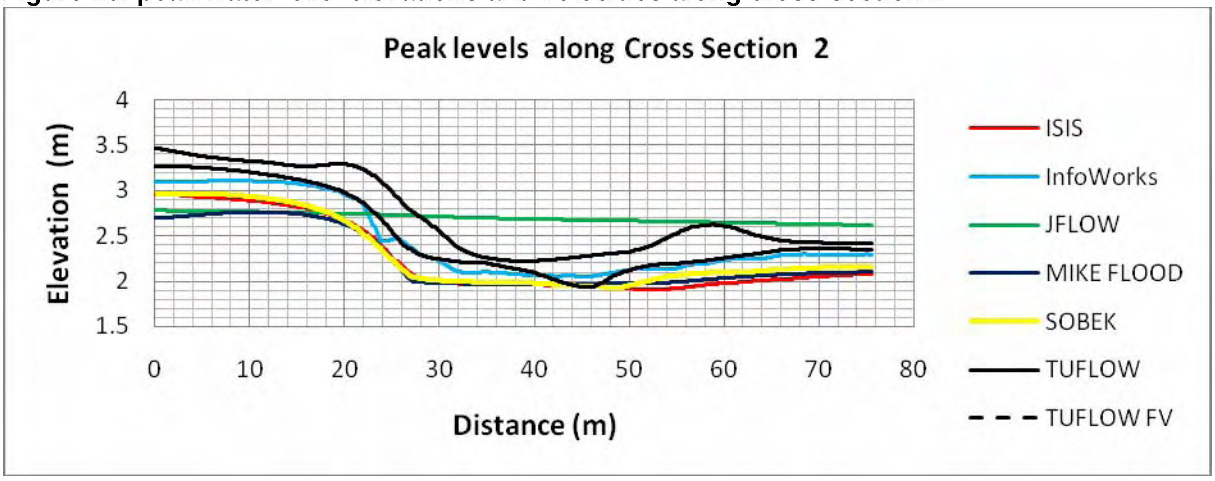
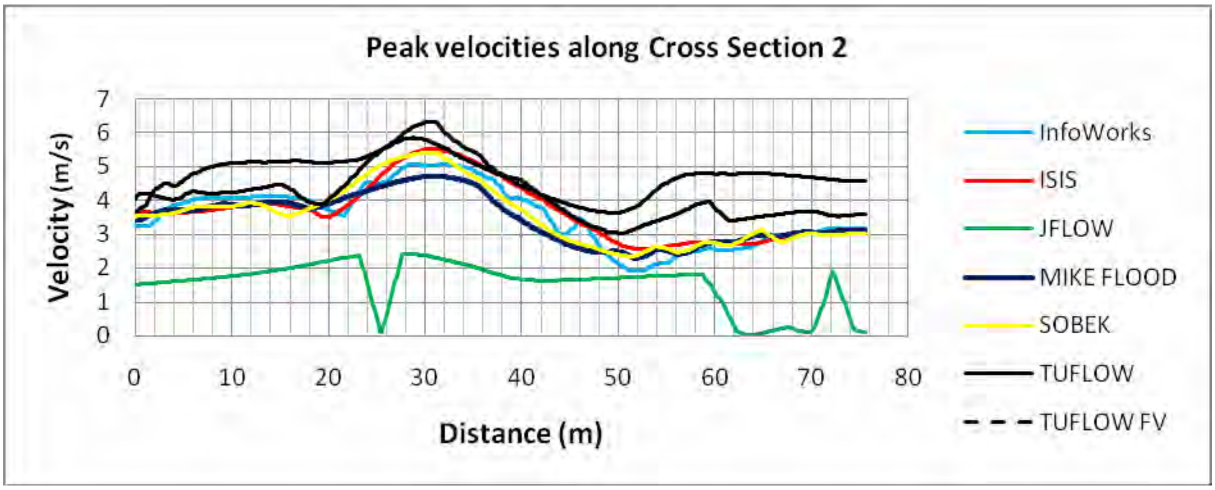


Figure 16: peak water level elevations and velocities along cross-section 2





Observations consistent with those made from the time series in Section 4.6.2.2 can be made from Figure 15 and Figure 16. In addition, Figure 15 shows that most models (except JFLOW-GPU) predicted a water surface consistent with the presence of a hydraulic jump between the abscissas 70m and 80m. The wake downstream from the building is also visible particularly in Figure 16. However TUFLOW predicted higher peak values, especially of levels in the area between the reservoir and the building (by up to ~40%), likely to be due to an initially slightly oscillatory behaviour in the solution (visible on some of the time series, e.g. at Point G4, although not commented on in Section 4.6.2.2).

Although JFLOW-GPU grids were supplied, these do not reflect the peaks calculated by JFLOW-GPU because of the excessively long output resolution (60s). Comments cannot be made.

4.6.2.4 Conclusions from Test 6B

All **full models** predicted similar results in terms of peak depths (within ~15 to 20%, for depths of up to ~3m) and peak velocities (within ~10% for velocities of up to 5 ms^{-1}, except downstream from the building). These differences are unlikely to be larger than the required accuracy of predictions for a problem of this type and scale in a practical application. Shock-capturing properties appear to be important in the prediction of peak values of velocity and depths, and critical transitions (predictions by models with these were more robust than by those without, e.g. TUFLOW).

The predictions by JFLOW-GPU where markedly different from the predictions by the full models due to the package's inability to simulate the sudden collapse of the water from the reservoir.

It was also confirmed by the developers of the other **simplified models** (Flood Risk Mapper, Flowroute, RFSM, UIM) that these are unlikely to produce usable results in an application similar to test 6B.

4.6.2.5 Summary of relevant technical information.

Note: 'N/A' in column (8) refers to the fact that the model does not account for eddy viscosity.

TEST 6B (1) Name	(2) Version (+ numeric. scheme)	(3) Hardware	(4) Multi-Proc.	(5) Grid (2m or 36000 elements)	(6) Time-stepping	(7) Run time (min)	(8) Eddy Visc. (m^2/s)
ANUGA	1.1beta_7501	Intel Mobile Core 2 Duo T7500 (Merom) 2.2 GHz RAM 2048 MB (DDR2)	no	36219 elements	adaptive	23.1	N/A
Flood Risk Mapper	*Not tested*						
FloodFlow	*Not tested*						
Flowroute	*Not tested*						
InfoWorks	RS 2D v10.5	Intel® Core™ i7 920 2.66GHz Quad Core 12 GB RAM (DDR3-1333)	OMP	35944 triangles	Adaptive	2.6	N /A
ISIS	3.3 TVD	Quad Intel Xeon DP 5050 @ 3.0 GHz, 4096MB RAM (FB-DDR2)	no	2m	0.02s	1186.3	0
JFLOW-GPU	*JFLOW-GPU*	Intel Pentium (R) D 2.8 GHz RAM 2.0 GB NVIDIA GeForce 9600 GT	Yes - GPU	2m	Avg 0.000701s	110.5	N/A
MIKE FLOOD	2009 incl. Serv. Pack 2	Intel Core 2 Duo CPU P8600 2.4 GHz RAM 4.00 GB	no	2m	0.25s	1.45	0.004
RFSM (Direct)	*Not tested*						
RFSM (Dynamic)	*Not tested*						
SOBEK	2.13	Intel i7 8 core CPU 2.66 GHz	no	2m	0.1s	16.9	0
TUFLOW	2010-01-AD-iSP	Intel Core 2 Duo T9800 2.93GHz RAM 4Gb	no	2m	0.2s	8.7	0.5 S + 0.1 C[29]
TUFLOW FV	0.107.0006 2nd order solution	Intel Xeon X5472 3.00GHz RAM 8Gb	Yes – 8 CPUs	31687 nodes	Typical time step: 0.01 – 0.02s	16.1 (2.8)	0.2 S[30]
UIM	*Not tested*						

[29] 0.5 (Smagorinsky) + 0.1 (Constant)

[30] 0.2 (Smagorinsky)

Other information provided:

ISIS: Simulations were also performed using the ADI scheme. Results were not stable or physically realistic.

JFLOW-GPU: "Although results have been supplied for Test 6B, JFLOW-GPU is not well-suited to instantaneous dam break problems (i.e. collapsing impounded volumes of water)."

"For Test 6B, it was necessary to use a more stringent condition for the time step than for the other test cases. This was necessary in order to preserve mass conservation and was as a result of the steep water surface gradient. This is reflected in the time step values and the run time for Test 6B."

Output of time series at 60s resolution, see Appendix B.4.

TUFLOW: Increasing the eddy viscosity and reducing the timestep reduces "noise" in the results during the initial dambreak, but does not overly change the results. The results use a Smagorinsky coefficient of 0.5 and a small timestep of 0.2s.

4.7 Test 7: River and floodplain linking

4.7.1 Introduction

This river and floodplain modelling test case (see Appendix A.7 for details and maps) consist of a ~7 km long by ~0.75 to ~1.75 km wide floodplain (composed of three distinct areas, Floodplains 1, 2 and 3). In the test the river Severn that flows through the site is modelled for a total distance of ~20km. Boundary conditions are a hypothetical inflow hydrograph and a downstream rating curve for the River Severn.

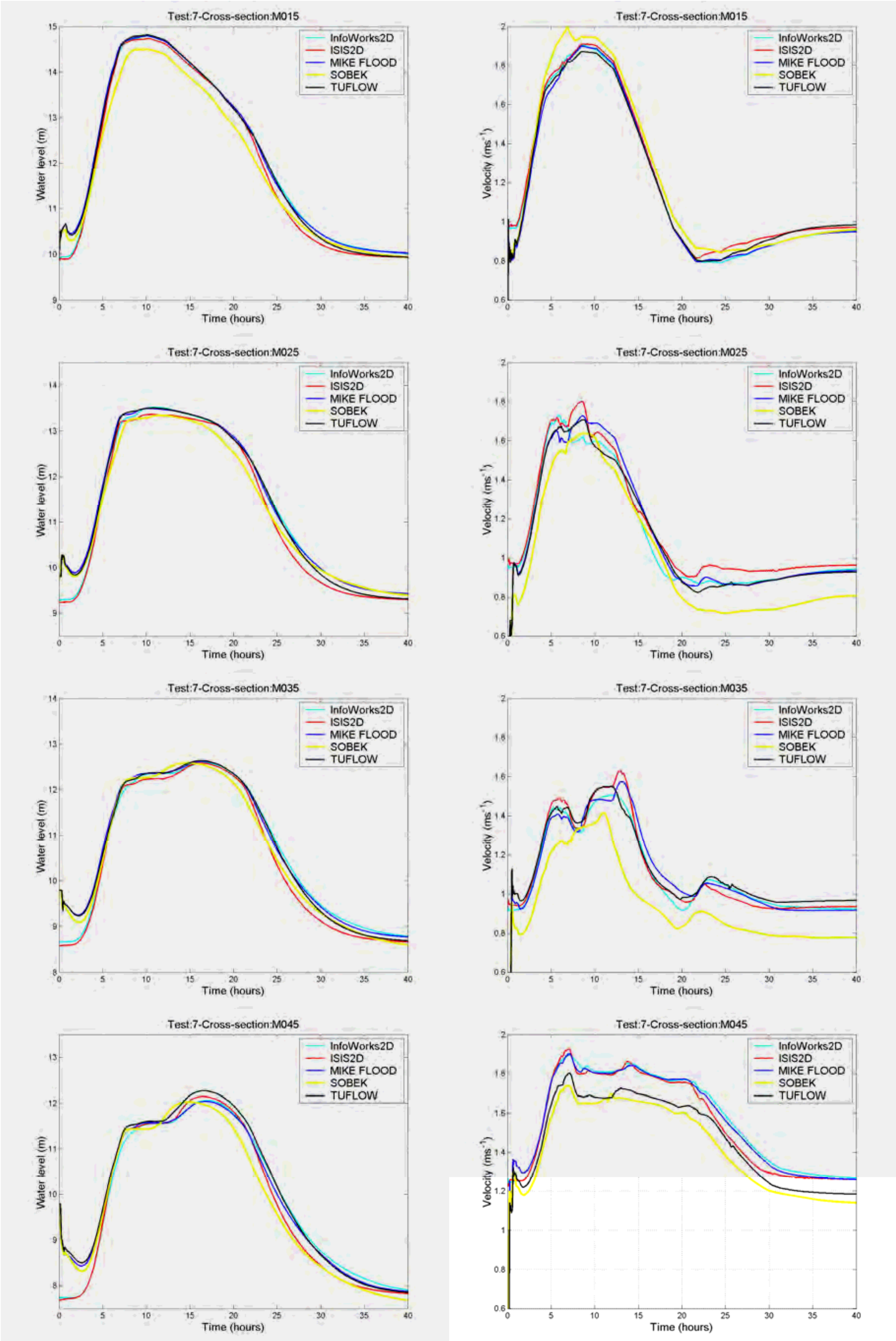
The objective of the test is to assess the package's ability to simulate fluvial flooding in a relatively large river, with floodplain flooding taking place as the result of river bank overtopping. The following capabilities are also tested: 1) the ability to link a river model component and a 2D floodplain model component, with volume transfer occurring by embankment/bank overtopping and through culverts and other pathways; 2) the ability to build the river component using 1D cross-sections; 3) the ability to process floodplain topography features supplied as 3D breaklines to complement the DEM[31].

4.7.2 River level and velocity time series

Water transfers between the river and the floodplains are governed by water level differences. It is therefore important to compare river water levels predicted by the models. 1D velocities are not taken into account in 1D river / 2D floodplain volume transfer calculations and are presented only as a means to help understanding differences in the predicted levels[32]. River bank elevations varied between ~13m at the top end of floodplain1 (near cross-section M024), to ~11.5m at the bottom end of floodplain3 (just upstream from M044). Water was able to flow into the South end of floodplain 1 for any river level above 10m through the 10m wide opening at M030 (this was suggested as an 'optional' feature in the original test specification, and was included in all models except the ISIS model). Water was able to flow through the Pool Brook culvert (at M033) into floodplain 3 at all times but a level of ~10.5m or higher was required for any significant flooding to happen.

[31] The breaklines provided were derived from the 1m DEM and were a 'vector' representation of important crest lines in the topography (including embankments). If participants were not able to implement the breaklines they were still expected to extract these crest elevations directly from the DEM.

[32] In the case of SOBEK the velocities plotted at cross-sections M025 and M035 are the resultants of the 2 velocity components calculated by the fully 2D model in this part of the river. Although SOBEK has the capability of linking 1D and 2D models both horizontally and vertically, in this test the developers adopted a fully 2D approach. In fully 2D models velocities calculated in the river influence the calculated overtopping discharges. However in square grid models due to the artificial representation of velocity along two orthogonal directions it is unclear whether this results in a more accurate representation of overtopping processes (than in combined 1D/2D models), particularly as a coarse grid must usually be used (if the floodplains are large a fine grid is likely to result in excessively long run times). The authors have been informed by Deltares of current R&D activities including the implementation of curvilinear grids for rivers.



The following observations can be made from the graphs above:

M015 is >3km upstream from the start of floodplain inundation and the level predictions are not affected by floodplain flooding in a significant manner. At the other three cross-sections the water level rise comes to a visible halt when significant overtopping starts to occur, stays almost constant for ~2 to ~5 hours and rises again by ~0.1 to ~0.6m when most floodplains have been filled. Close inspection of the graphs reveal that river levels reach the relevant bankfull levels (not all visible on graphs above):

- At M025, within a ~25mins window according to MIKE FLOOD, TUFLOW, ISIS and InfoWorks, and ~25mins after these according to SOBEK.
- At M035 within a ~30min range
- At M045 within a ~30min range

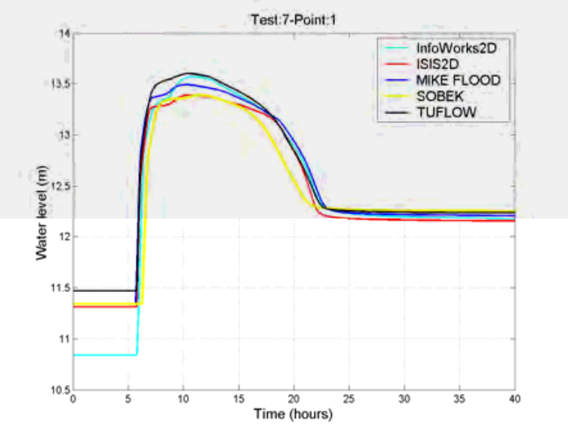
Peak water levels predicted were:

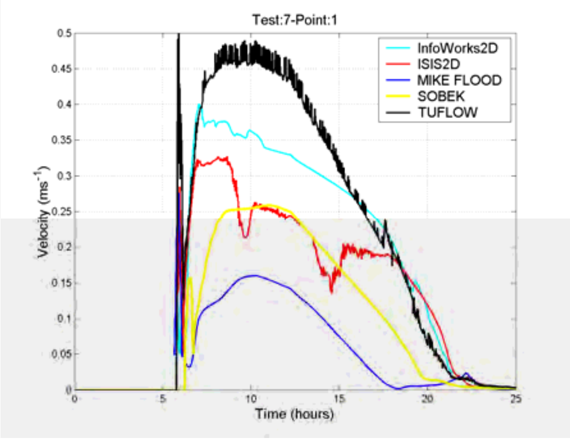
- At M025, ~13.5m ±0.02m according to InfoWorks, TUFLOW and MIKE FLOOD; 13.35m ±0.01m according to ISIS and SOBEK.
- At M035, ~12.63m ±0.01m according to TUFLOW and MIKE; 12.58m ±0.01m according to InfoWorks, ISIS and SOBEK.
- At M045, ~12.27m according to TUFLOW; 12.14m according to ISIS; 12.04m ±0.01m according to InfoWorks, MIKE and SOBEK.

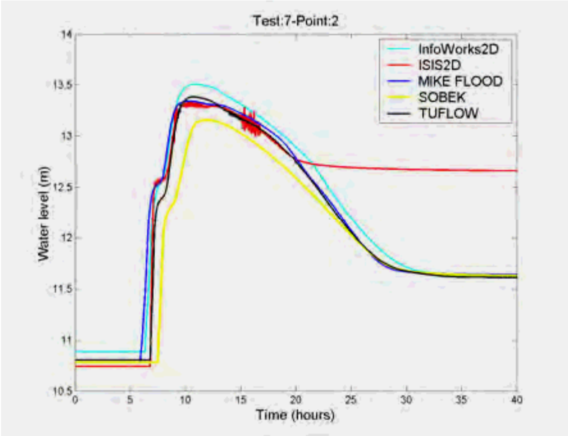
The relatively large differences observed above can be broadly explained by the different schemes for linking the 1D and 2D models and different model constructions. Exchange processes between river and floodplain are complex and this is reflected in the models. Small differences upstream affect local inundation processes, which in turn affect river levels and inundation processes further downstream. A “cascading” effect operates, which makes it unlikely that models will return identical results.

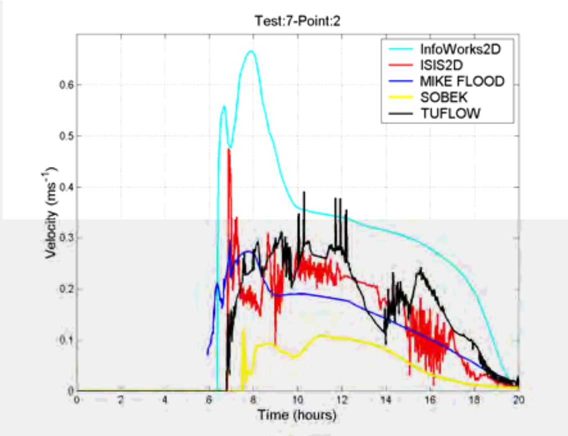
4.7.3 Water levels (and velocities) on floodplains

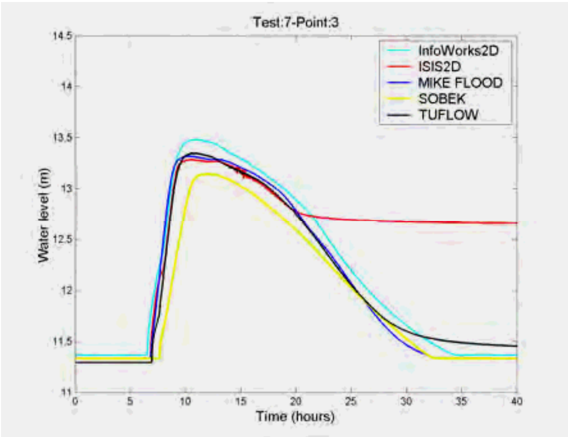
Floodplain 1

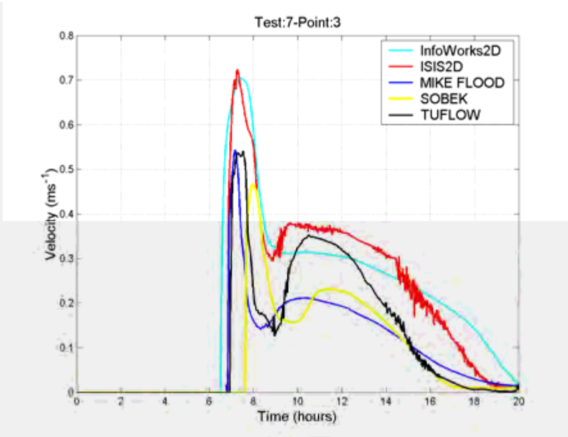


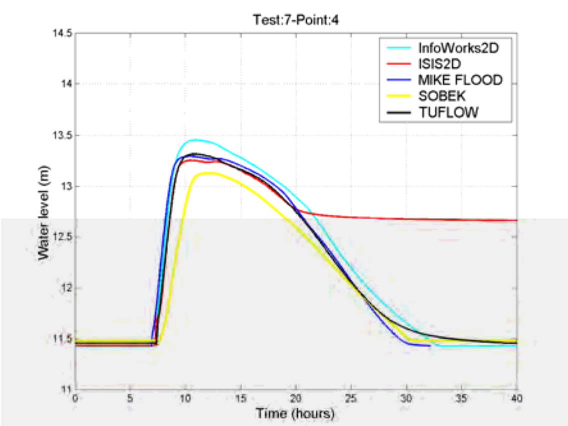


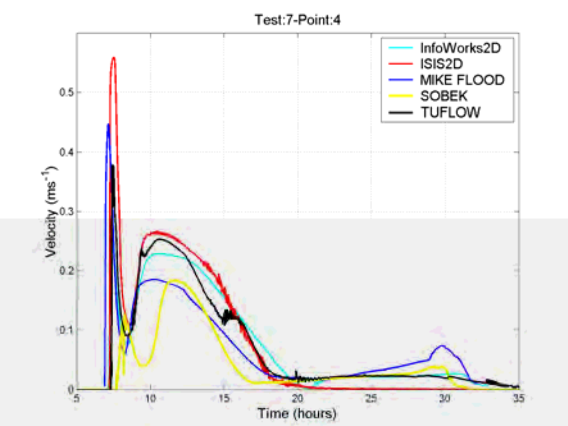

Test:7-Point:2
Water level (m)
Time (hours)
InfoWorks2D
ISIS2D
MIKE FLOOD
SOBEK
TUFLOW

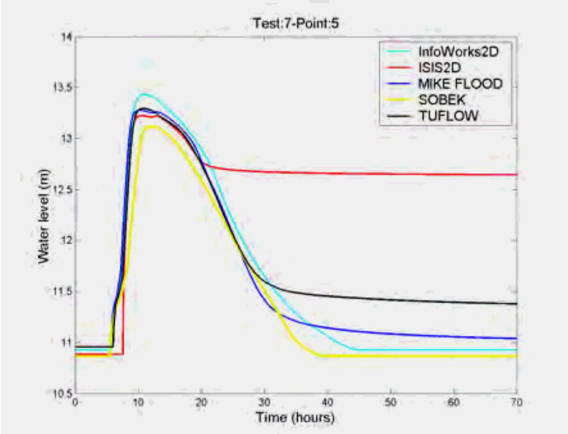

Test:7-Point:2
Velocity (ms-1)
Time (hours)
InfoWorks2D
ISIS2D
MIKE FLOOD
SOBEK
TUFLOW

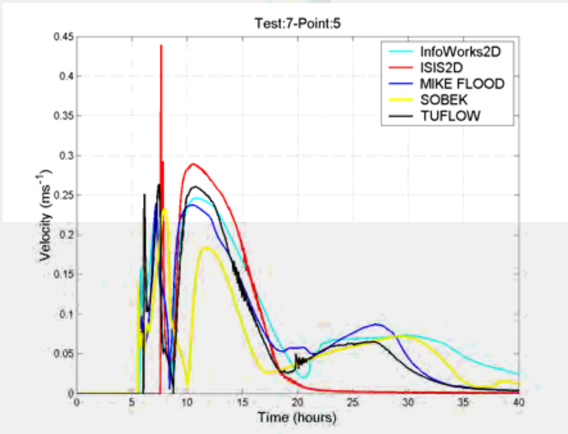

Test:7-Point:3
Water level (m)
Time (hours)
InfoWorks2D
ISIS2D
MIKE FLOOD
SOBEK
TUFLOW

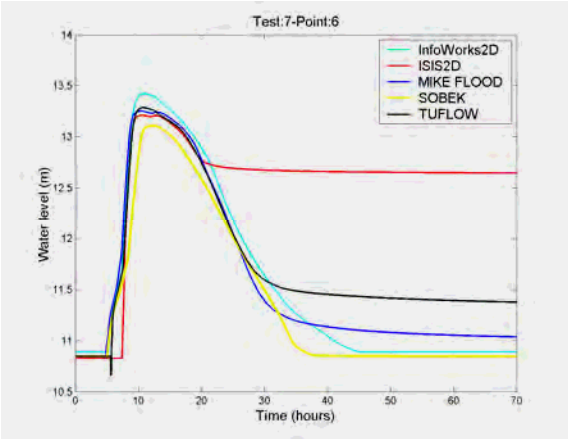

Test:7-Point:3
Velocity (ms-1)
Time (hours)
InfoWorks2D
ISIS2D
MIKE FLOOD
SOBEK
TUFLOW

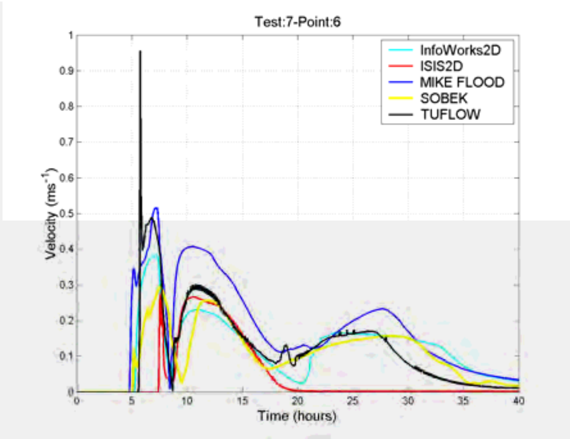

Test:7-Point:4
Water level (m)
Time (hours)
InfoWorks2D
ISIS2D
MIKE FLOOD
SOBEK
TUFLOW


Test:7-Point:4
Velocity (ms-1)
Time (hours)
InfoWorks2D
ISIS2D
MIKE FLOOD
SOBEK
TUFLOW


Test:7-Point:5
Water level (m)
Time (hours)
InfoWorks2D
ISIS2D
MIKE FLOOD
SOBEK
TUFLOW


Test:7-Point:5
Velocity (ms-1)
Time (hours)
InfoWorks2D
ISIS2D
MIKE FLOOD
SOBEK
TUFLOW





Point 1:

Point 1 lies in a low depression in an area of FP 1 not protected by flood embankments. Predicted peak levels and arrival times are consistent with the 1D predictions at river cross-sections nearby. The final level after 'dewatering' should theoretically be ~12.13m, ie. the level of the lowest point along the river bank breakline provided, which ISIS and InfoWorks predicted accurately, while others predict a level up to ~12cm too high. The fact that this low point was localised and between two higher points ~70m apart (a small distance compared to the grid resolution of 20m), may explain these discrepancies, although an accurate analysis could only be done in light of the details of modelling approaches used in the various packages.

Points 2,3,4,5,6:

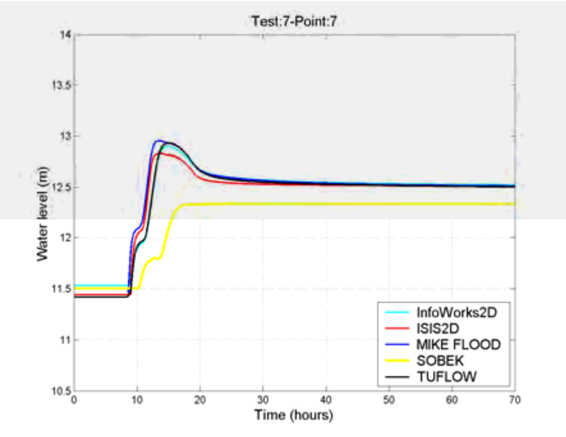
In the rest of the floodplain (Points 2 to 6) the peak levels predicted are all consistent with the presence of a single almost horizontal water body (with a ~5cm level difference at most between Point 2 at the North end and point 6 at the South end, depending on the model) at mean elevation ~13.45m for InfoWorks, ~13.3m for MIKE, TUFLOW and ISIS, and 13.15m for SOBEK (see also Figure 18). This is generally higher than the embankment elevations (~13.1m at North end to ~12.8m at South end of floodplain). These peak levels are also lower than river levels at the North end (near M025) and higher at the South end near cross-section M030 (most probably, as the levels at this location can only be estimated by interpolation from available data at M025 and M035). The relatively large differences in these predicted peak levels are not only due to differences in river levels (than which they are larger), they are also caused by differences in the approaches used to a) model overtopping (which may include crucial parameters such as discharge coefficients,etc.), b) the implementation of the embankment crest elevations, and c) modelling of the 10m opening near cross-section M030.

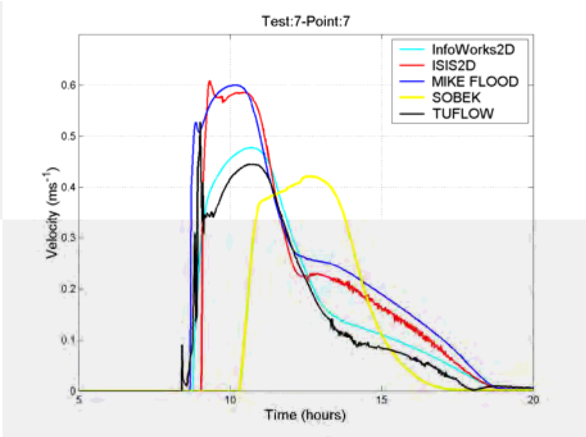
At Points 5 and 6 an early arrival of the flood can be observed with all models except the ISIS model explained by the flow through the 10m opening in the embankment near M030, which dominated local flooding in the first ~2hours (this feature was not included in the ISIS model because it was specified as 'optional' in the first version of the specification).

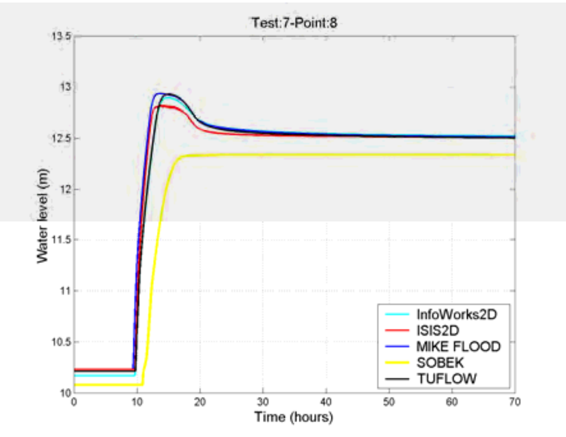
Dewatering at Points 2,3,4,5,6:

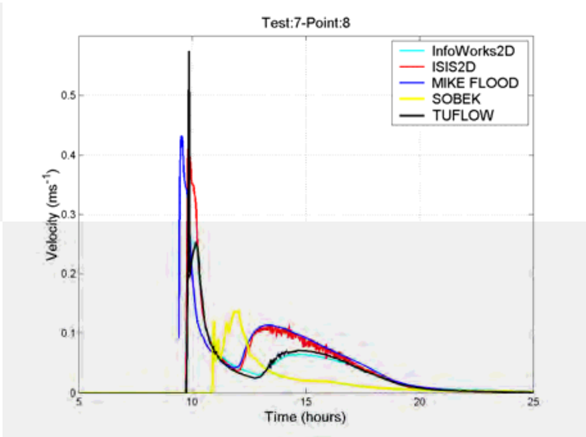
With all models except ISIS, the floodplain eventually dries out (through the opening at M030), but water remains in some depressions (such as at Point 5). With ISIS the levels decreased back to the lowest level along the embankment (~12.62m) which is correctly predicted.

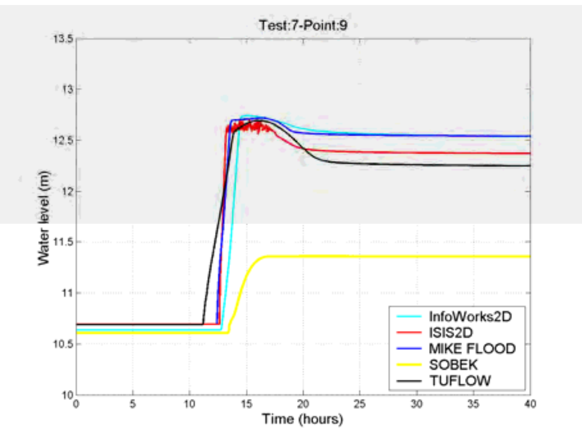
Floodplain 2

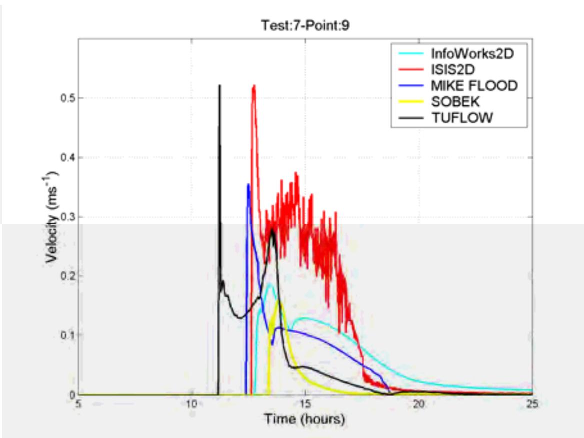












Points 7 and 8:

Observations in Floodplain 2 differ from the ones in Floodplain 1 in that calculated embankment overtopping depths were generally smaller (this can be estimated from the river levels calculated), resulting in larger discrepancies between models in the prediction of overtopping discharges and of the duration during which overtopping happened.

It can be observed in the graphs for Points 7 and 8 that every model predicted identical levels at these two points from the time of the peak onwards, reflecting the fact that both points were part of a single water body (see also Figure 18). All models except SOBEK predicted the floodplain level to rise above the embankment elevations and become controlled by river levels. According to the SOBEK model, the floodplain level stopped rising when overtopping stopped, before reaching the river level or the lowest point along the embankment crest (at elevation ~12.50m).

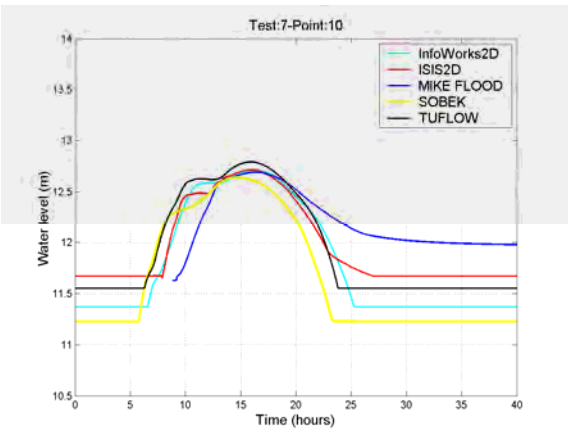
The final water level (after the recession) in all simulations, except the SOBEK simulation (for the reason given above) is predicted correctly as being equal to ~12.5m (the lowest point along embankment). No drainage pathway through culverts etc. were to be included for floodplain 2.

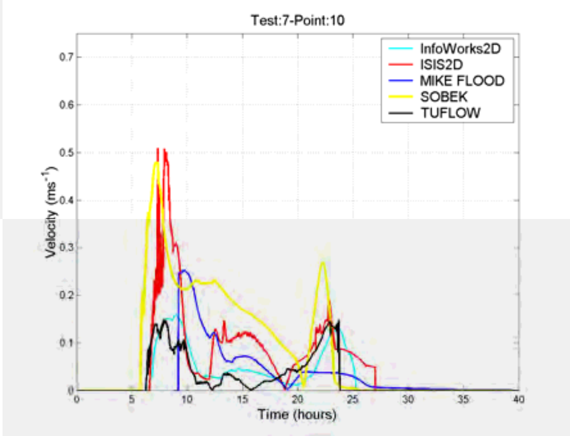
It is emphasised that the different behaviour in the SOBEK results above should not be interpreted as a shortcoming in the SOBEK model or in the numerical solver itself. It is likely to reflect the fact that the SOBEK model was entirely 2D. However it is possible that calculated river/floodplain transfer discharges were very different even between the InfoWorks, ISIS, MIKE and TUFLOW models, even if this did not result in differences in floodplain peak levels during the peak of the flood larger than the differences in the calculated river levels.

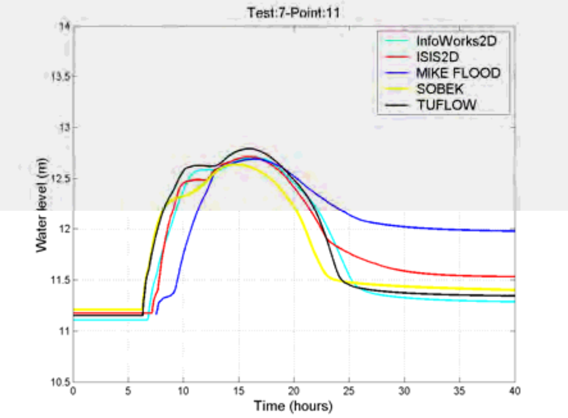
Point 9:

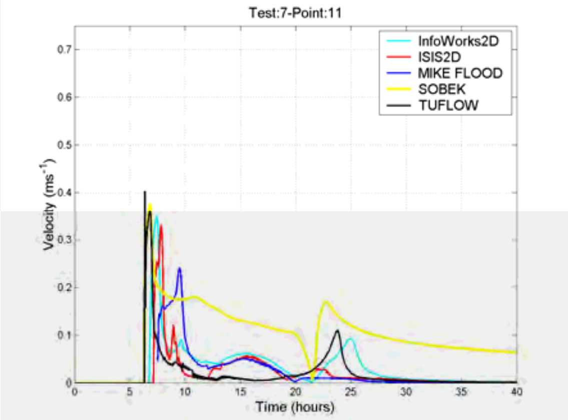
Flooding could only occur at this location if the river level around cross-section M035 rose to above ~12.49m (a low point along the embankment to the South). This was only the case by a small amount (~0.08 to ~0.13m depending on the model). Observations are similar to those at Points 7 and 8 in that all models except SOBEK predicted the peak level to become controlled by the river level. The final level after dewatering (governed by the same low point) was correctly predicted by InfoWorks and MIKE, but underestimated by ISIS (by ~0.12m) and TUFLOW (by ~0.25m).

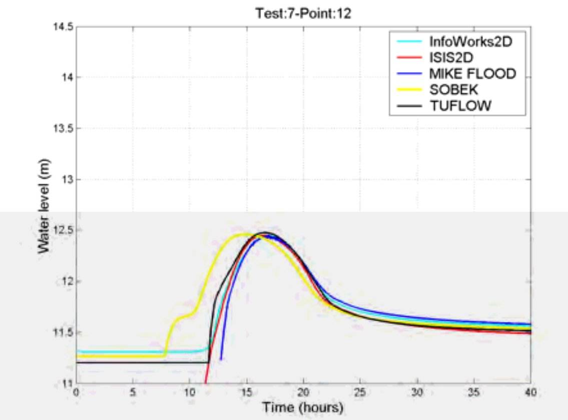
Floodplain 3

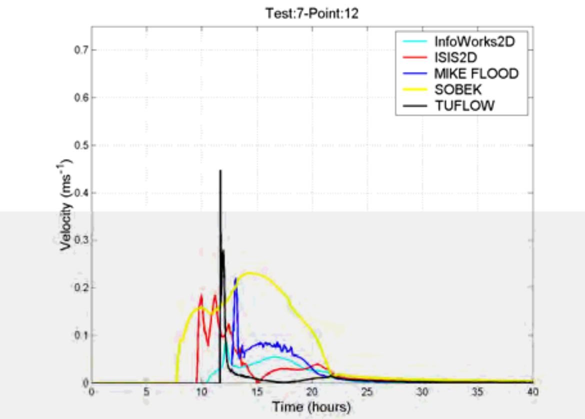


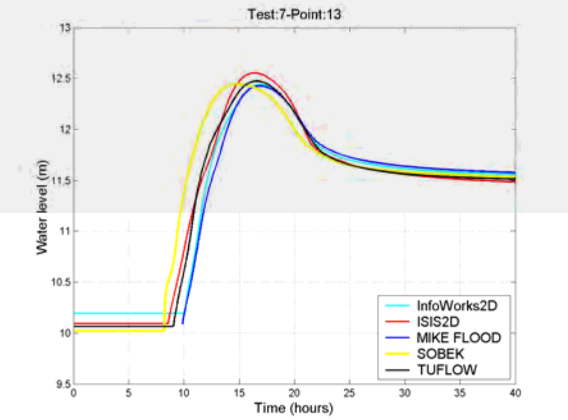


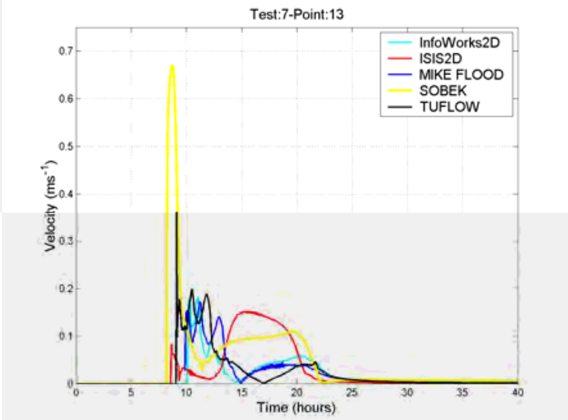

Test:7-Point:11
Water level (m)
Time (hours)
InfoWorks2D
ISIS2D
MIKE FLOOD
SOBEK
TUFLOW

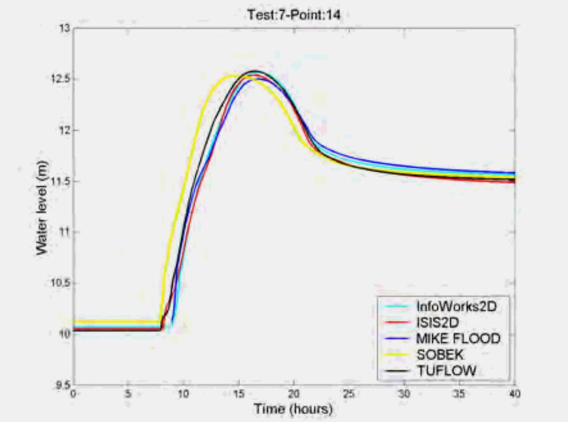

Test:7-Point:11
Velocity (ms-1)
Time (hours)
InfoWorks2D
ISIS2D
MIKE FLOOD
SOBEK
TUFLOW

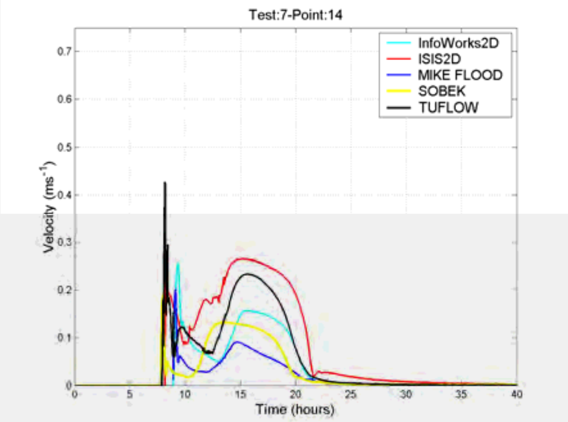

Test:7-Point:12
Water level (m)
Time (hours)
InfoWorks2D
ISIS2D
MIKE FLOOD
SOBEK
TUFLOW

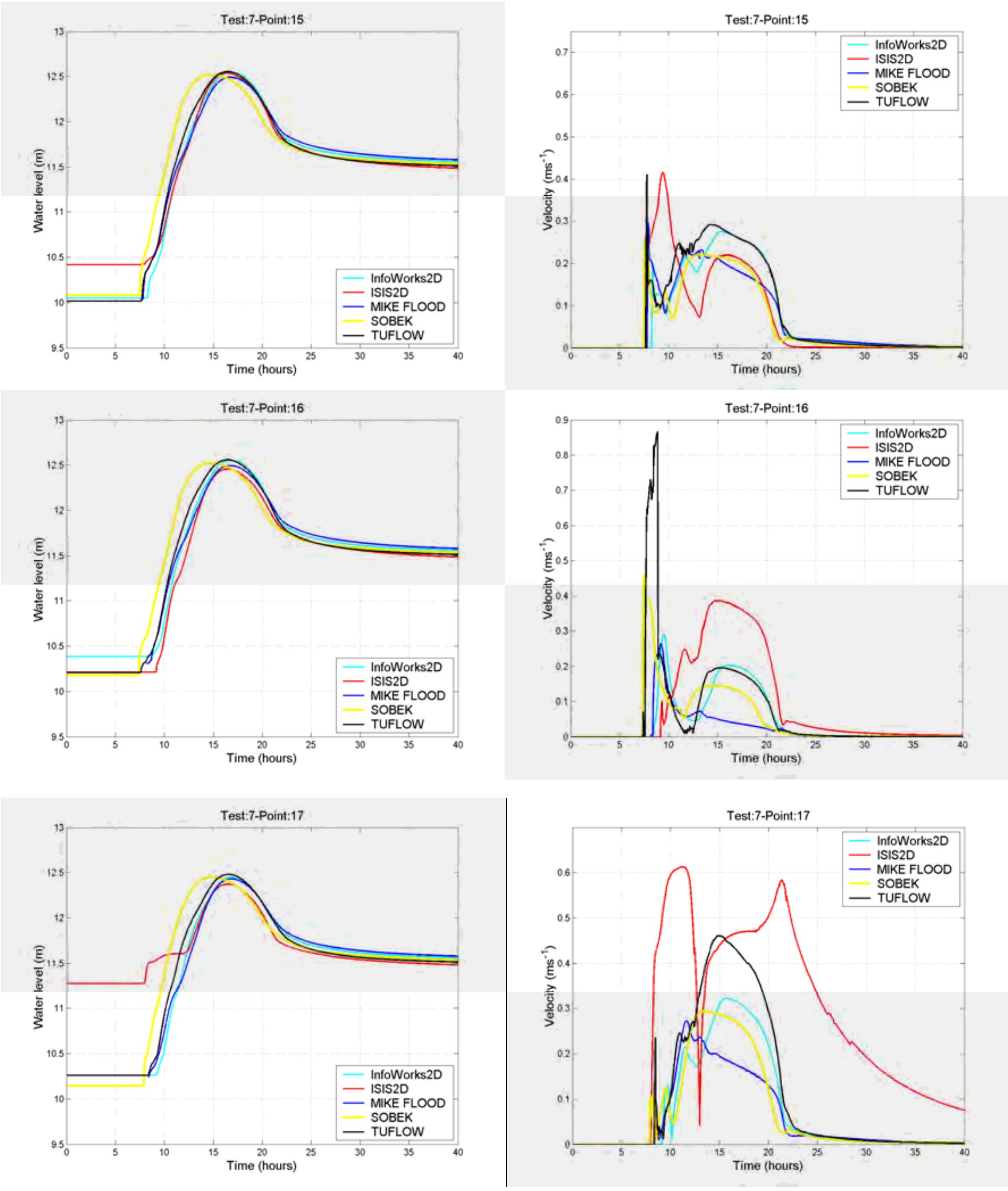

Test:7-Point:12
Velocity (ms-1)
Time (hours)
InfoWorks2D
ISIS2D
MIKE FLOOD
SOBEK
TUFLOW


Test:7-Point:13
Water level (m)
Time (hours)
InfoWorks2D
ISIS2D
MIKE FLOOD
SOBEK
TUFLOW


Test:7-Point:13
Velocity (ms-1)
Time (hours)
InfoWorks2D
ISIS2D
MIKE FLOOD
SOBEK
TUFLOW


Test:7-Point:14
Water level (m)
Time (hours)
InfoWorks2D
ISIS2D
MIKE FLOOD
SOBEK
TUFLOW


Test:7-Point:14
Velocity (ms-1)
Time (hours)
InfoWorks2D
ISIS2D
MIKE FLOOD
SOBEK
TUFLOW



Points 10 and 11:

These points lie in the area to the west of Upton-upon-Severn where flood levels are expected to be similar to those in the river levels with an almost immediate response, because of the very large culvert (modelled) allowing the Pool Brook (not modelled) to flow into the Severn. Considering the timing of the flood as predicted in the river by the models, they should have experienced a level of at least 12m by t≈7.5hr. This was correctly observed with SOBEK and TUFLOW. A delay of ~1 hr was observed with InfoWorks and ISIS, of ~3hr with MIKE, suggesting shortcomings in the modelling of the culvert.

Points 12 to 17:

In this area the floodplain is not defended and the banks are natural, albeit with a slight natural slope (downward) away from the river. The observations made at all these points are very similar, and the curves show that they are all part of a single body of water during most of the flood (see also Figure 18). Peak levels on the floodplain are controlled by peak levels in the river (larger overtopping depths occurring over long distances and durations, allowing the full floodplain capacity to be occupied by water). There were differences however in arrival times within a range of up to ~2.5hours, with SOBEK prediction of peaks arriving earlier than others. SOBEK also predicted a much earlier initial rise at point 12, which was due to the approach used to model openings through the road embankments to the North, allowing the flood to arrive earlier through this route. The significantly different “dry” elevations applied by ISIS at points 15 and 17 are likely to be due to local topography effects. The final elevation of ~11.5m, due to a low point in the river bank crest elevation, is predicted by all models within ~0.05m.

All floodplains

Velocities

There are very significant discrepancies in the predicted velocities as can be observed in all velocity plots. These differences can be explained from two different effects:

- At the beginning of the flood, immediately after overtopping of the embankment or river banks, sharp peaks can be observed as the flow finds its way along floodplain slopes. The magnitude of this peak is heavily dependent on a) predicted overtopping discharges; b) the rate of change in time of these discharges; c) the ability of the models to handle the predictions of highly transient flows (TUFLOW predicted a sharp peak at several points). The magnitude of this peak value is therefore unlikely to be consistently predicted beteween models. This is also likely to affect any peak velocity grid output from a model.
- At later stages a quasi-steady flow lasting several hours often occurs, as a small head difference exists between the North and the South end of the floodplains. As commented on above discrepancies exist between models (for various reasons detailed above) in the magnitude of these slopes, resulting in discrepancies in the calculated velocities.

The above comments suggest that velocity predictions are unlikely to be accurate in river / floodplain models such as the one in this test. This also applies to any peak velocity grids output from the models, as confirmed in Section 4.7.4.

4.7.4 Output in gridded format

Figure 17: Peak velocities predicted in test 7.

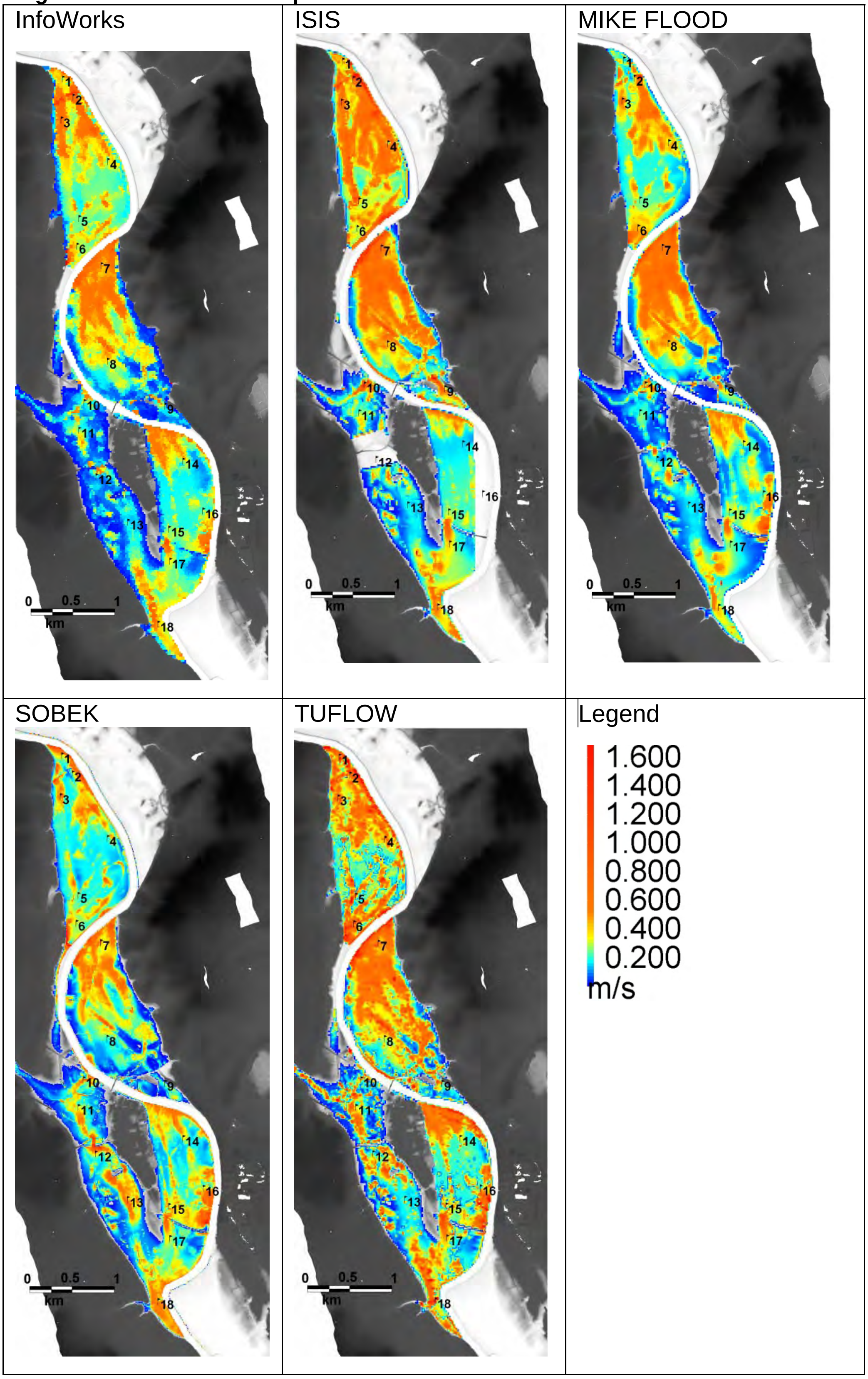
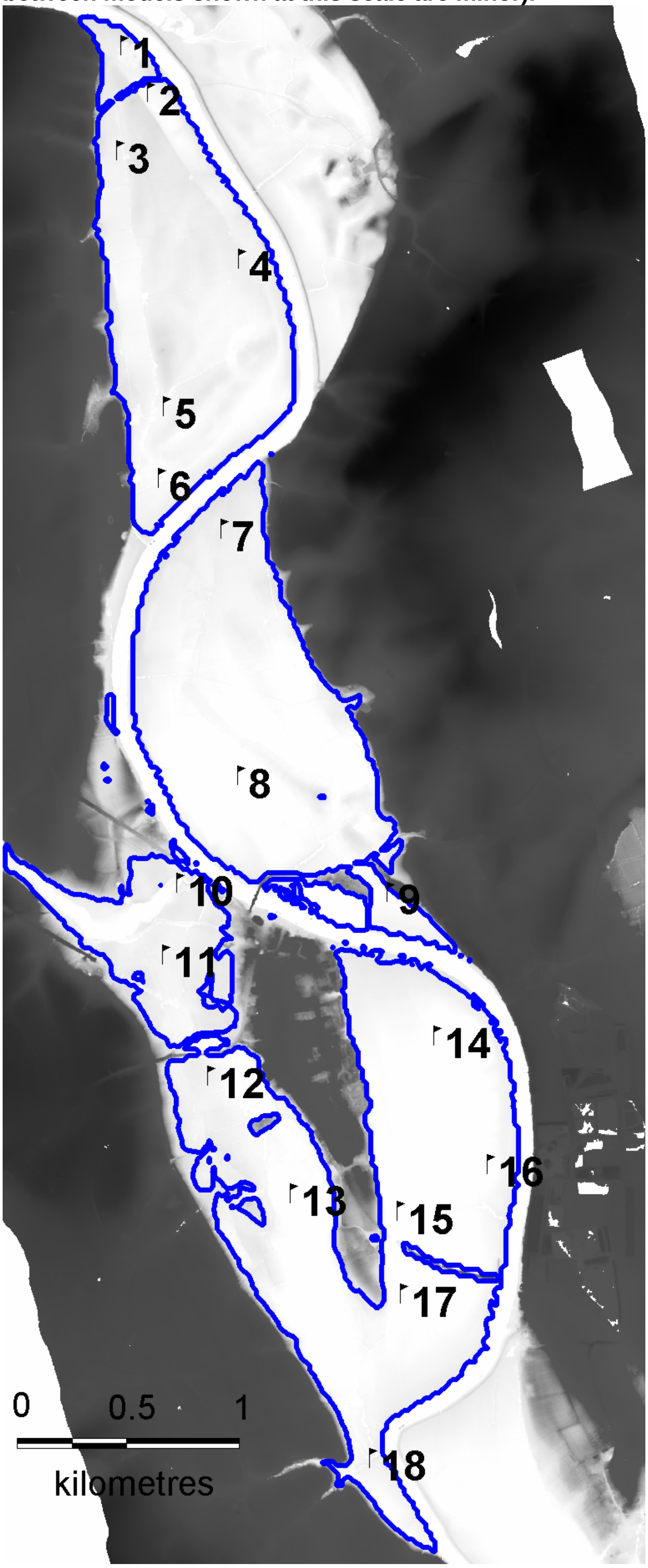


Figure 18: 0.5m contours of peak depths, as predicted by one of the models (differences between models shown at this scale are minor).



The main comment that can be made from Figure 17 is that predicted peak velocities are significantly different between the five models, confirming comments in Section 4.7.3. Peak velocity mapping is therefore likely to be only indicative for this type of problem.

4.7.5 Conclusions and discussion from Test 7

All packages participating in test 7 have demonstrated their ability to implement linked 1D river / 2D floodplain modelling[33]. This functionality is not yet supported in the current versions of Flood Risk Mapper, Flowroute, JFLOW-GPU, RFSM, TUFLOW FV, UIM and ANUGA.

Unlike in other tests the discussion above has not identified a high level of consistency in the results produced by the various models. Large discrepancies between models are observed, reflecting the physics of a fluvial flood event of this type, where:

- River and floodplain dynamics are complex. Exchanges of water affect river levels which in turn affect exchanges downstream, even upstream in subcritical river flows, resulting in complicated “cascading” propagation of difference between models.
- These exchanges depend critically on river bank or embankment overtopping discharges, and on the flow through structures,
- Peak velocities on floodplains depends on overtopping discharges, the flow through structures and the rate at which these change in time

Accurate modelling of these exchange processes is therefore crucial to the accurate prediction of flood hazard on floodplains where linked 1D/2D models is used. This includes the need to accurately implement the geometry of critical structures such as embankments, or even natural river banks (where any errors must be small compared to typical overtopping depths, which are often as small as ~0.1m).

Although the floodplain topography and dimensions of structures were specified, participants used different modelling approaches and parameters to model overtopping (these were not specified and robust modelling techniques with appropriate guidance on parameterisation do not exist). In addition there is evidence that the participants were not always able to implement the correct structure dimensions or the correct elevations along river banks and embankments. Errors concerning these were often comparable to, if not larger than overtopping depths.

In all cases the models implemented horizontal linking so in this report vertical linking, which may offer advantages, has not been tested.

[33] In the case of SOBEK, while a 1D/2D link is also supported, Deltares have supplied results from a model where the river was part of the 2D mesh, their preferred approach for this type of problem.

4.7.6 Summary of relevant technical information

TEST 7 (1) Name	(2) Version (+ numeric. scheme)	(3) Hardware	(4) Multi-Proc.	(5) Grid (20m or 16700 elements)	(6) Time-stepping	(7) Run time (min)
ANUGA	*Not tested*					
Flood Risk Mapper	*Not tested*					
FloodFlow	W.12.0 Beta ADI	Intel(R) Core™ 2 CPU 6400 2.13GHz RAM 2GB	no	20m	Adaptive	50
Flowroute	*Not tested*					
InfoWorks	RS 2D v10.5	Intel® Core™ i7 920 2.66GHz Quad Core 12 GB RAM (DDR3-1333)	OMP	20994 triangles	Adaptive 0.1s to 60s	87
ISIS	3.3 ADI	Quad Intel Xeon DP 5050 @ 3.0 GHz, 4096MB RAM (FB-DDR2)	Partial, see section 4.0.3	20m	2s	51
JFLOW-GPU	*Not tested*					
MIKE FLOOD	2009 incl. Serv. Pack 2	Intel Core 2 Quad CPU Q9450 2.66 GHz RAM 3.48 GB	no	20m	4s	11.27
RFSM (Direct)	*Not tested*					
RFSM (Dynamic)	*Not tested*					
SOBEK	2.13	Intel i7 8 core CPU 2.66 GHz	no	10m	10s	194.9
TUFLOW	2010-01-AD-iSP	Intel Core 2 Duo T9800 2.93GHz RAM 4Gb	no	20m	10s	9.9
TUFLOW FV	*Not tested*					
UIM	*Not tested*					

Reasons for undertaking Test 7 at a different grid resolution:

SOBEK: 2D modelling of river with 20m grid is too coarse. 2D modelling of river with 10m grid is more appropriate.

Other information:

There was room for the modellers' own initiative in Test 7 on how to model a number of features. All information provided by the participants on modelling approaches is included in Appendix C.

4.7.7 Test 8A: Rainfall and point source surface flow in urban areas

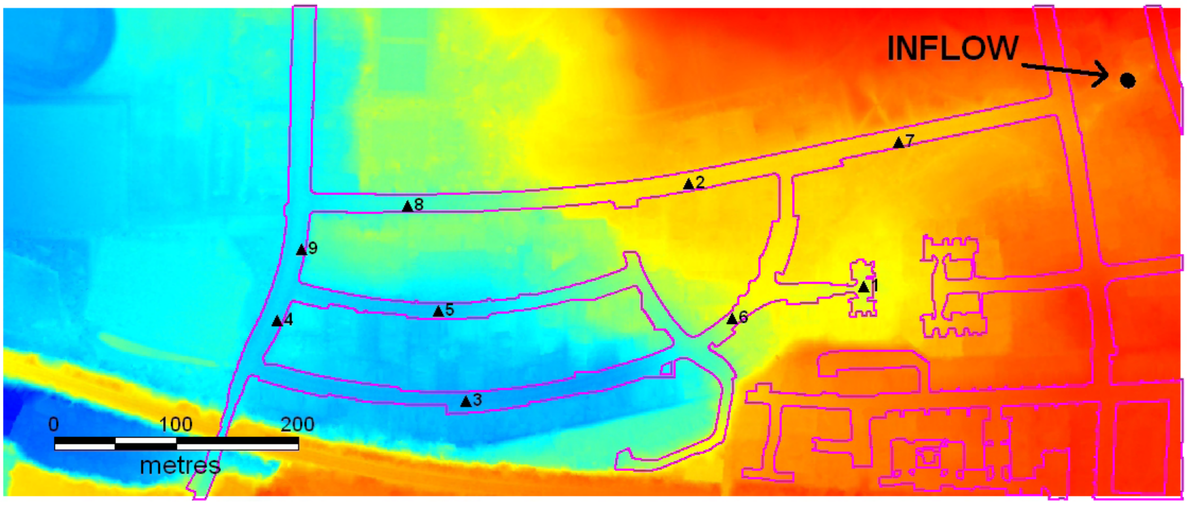
4.7.8 Introduction

The modelled area (see Appendix A.8 for details) is approximately 0.4 km by 0.96 km and is shown in Figure 19. The flood is assumed to arise from two sources:

- a uniformly distributed rainfall event (peaking at 400mm/hour over a time base of 3 mins), applied to the modelled area.

- a point source at the location represented in Figure 19, occurring over a time base of ~15mins, with a peak at 5 m^3/s occurring ~35mins after the rainfall event.

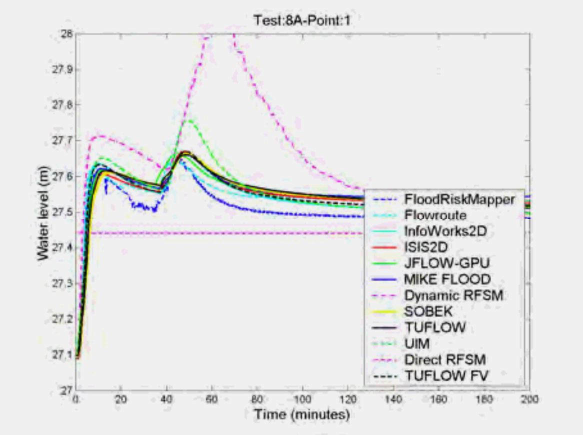
Figure 19: DEM used, with the location of the point source. Purple lines: outline of roads and pavements. Triangles: output point locations.

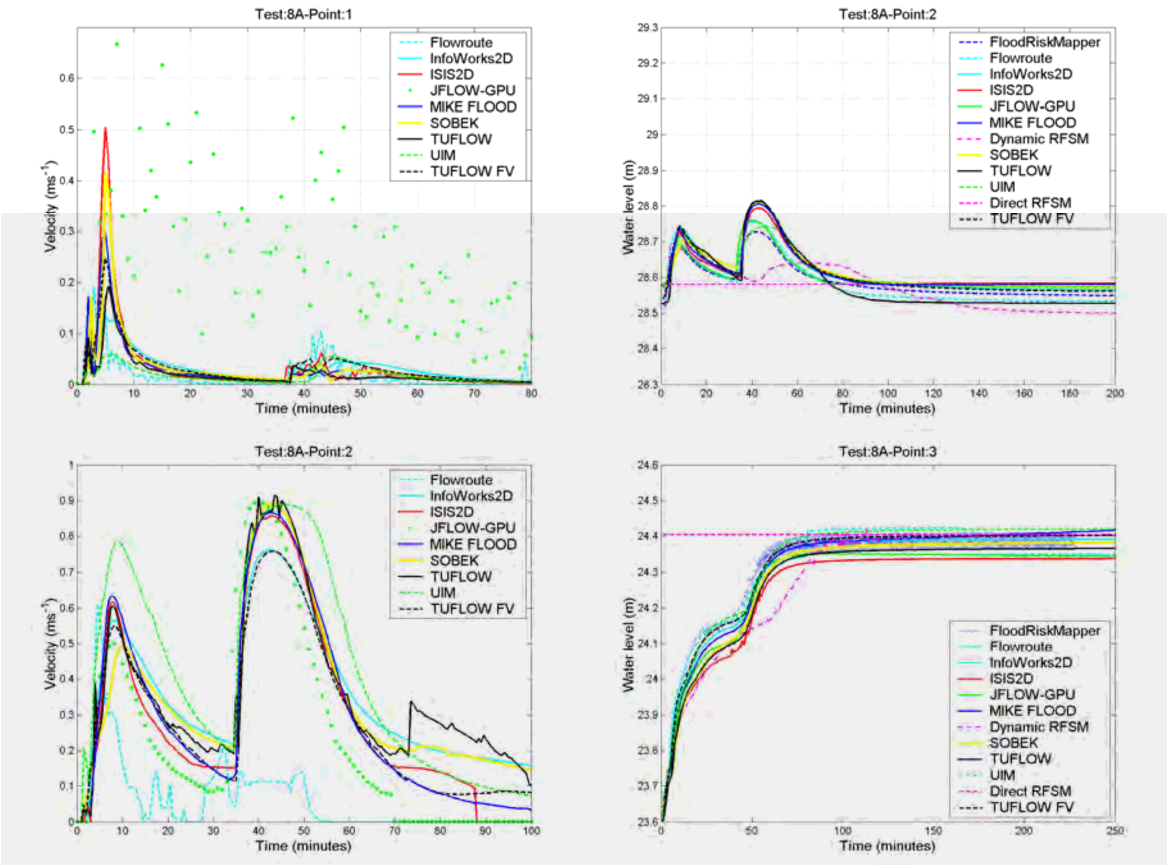


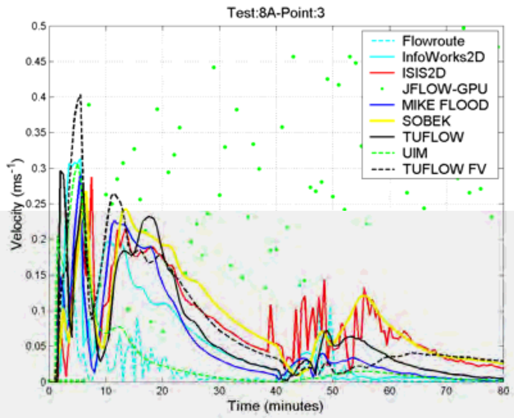
This tests the package's capability to simulate shallow inundation originating from a point source and from rainfall applied directly to the model grid, at relatively high resolution.

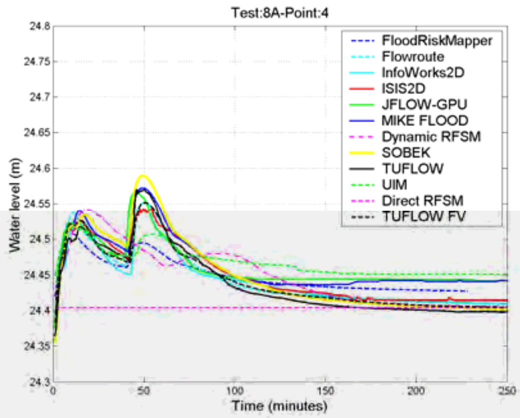
4.7.9 Water level and velocity time series

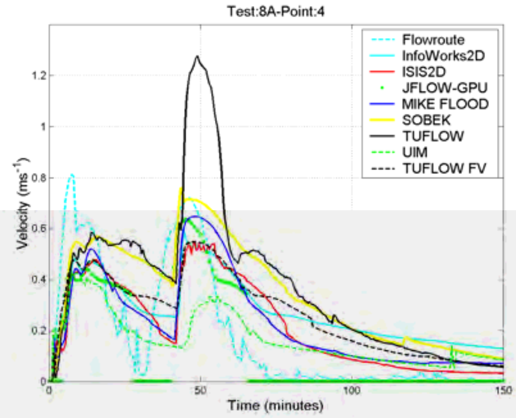
Note: Axes have been optimised for each graph and are not all identical.

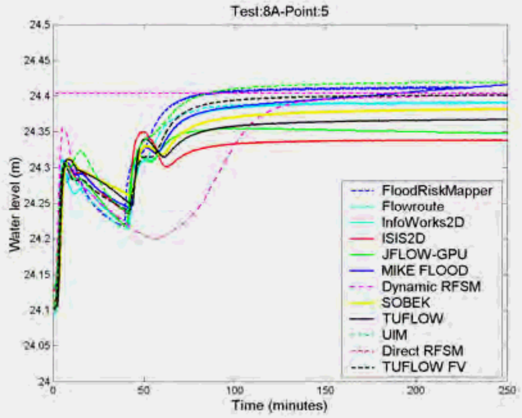


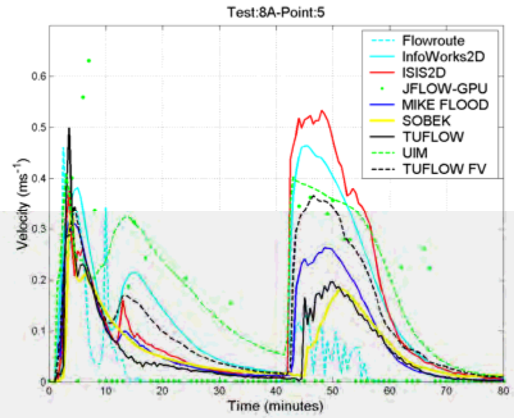


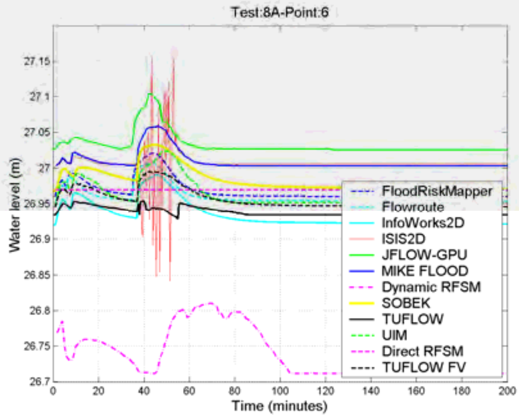

Test:8A-Point:3
Velocity (ms-1)
Time (minutes)
Flowroute
InfoWorks2D
ISIS2D
JFLOW-GPU
MIKE FLOOD
SOBEK
TUFLOW
UIM
TUFLOW FV

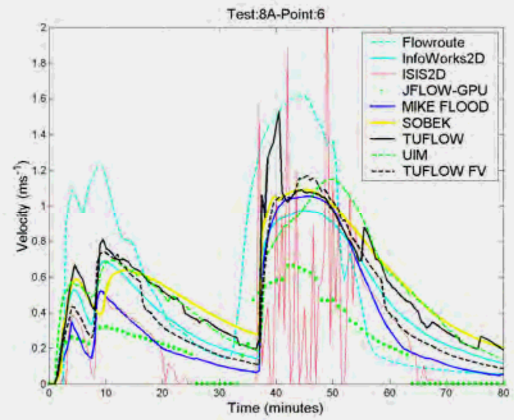

Test:8A-Point:4
Water level (m)
Time (minutes)
FloodRiskMapper
Flowroute
InfoWorks2D
ISIS2D
JFLOW-GPU
MIKE FLOOD
Dynamic RFSM
SOBEK
TUFLOW
UIM
Direct RFSM
TUFLOW FV

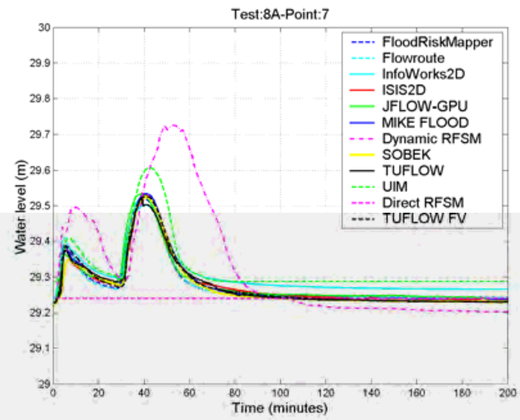

Test:8A-Point:4
Velocity (ms-1)
Time (minutes)
Flowroute
InfoWorks2D
ISIS2D
JFLOW-GPU
MIKE FLOOD
SOBEK
TUFLOW
UIM
TUFLOW FV

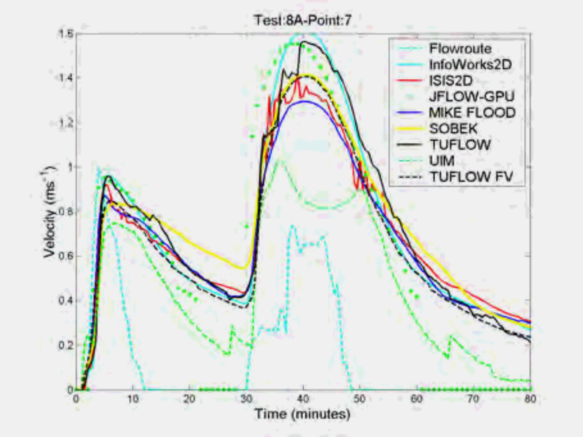

Test:8A-Point:5
Water level (m)
Time (minutes)
FloodRiskMapper
Flowroute
InfoWorks2D
ISIS2D
JFLOW-GPU
MIKE FLOOD
Dynamic RFSM
SOBEK
TUFLOW
UIM
Direct RFSM
TUFLOW FV

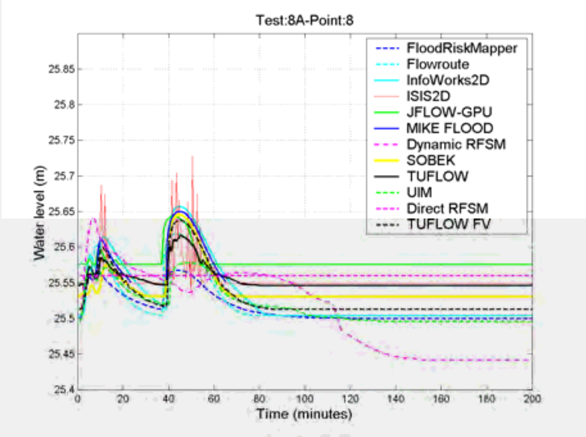

Test:8A-Point:5
Velocity (ms-1)
Time (minutes)
Flowroute
InfoWorks2D
ISIS2D
JFLOW-GPU
MIKE FLOOD
SOBEK
TUFLOW
UIM
TUFLOW FV

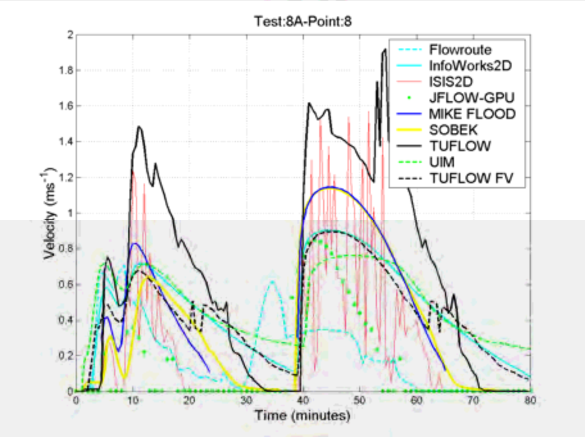

Test:8A-Point:6
Water level (m)
Time (minutes)
FloodRiskMapper
Flowroute
InfoWorks2D
ISIS2D
JFLOW-GPU
MIKE FLOOD
Dynamic RFSM
SOBEK
TUFLOW
UIM
Direct RFSM
TUFLOW FV

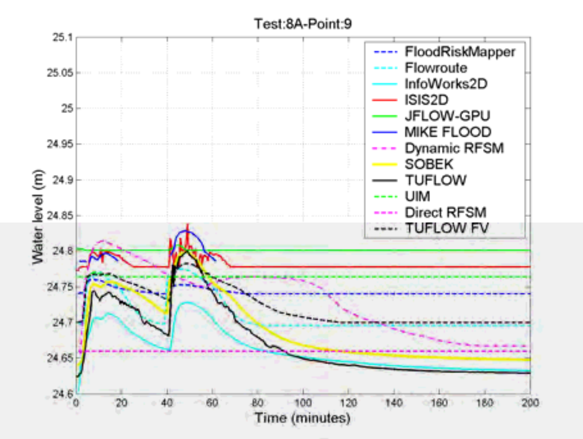

Test:8A-Point:6
Velocity (ms-1)
Time (minutes)
Flowroute
InfoWorks2D
ISIS2D
JFLOW-GPU
MIKE FLOOD
SOBEK
TUFLOW
UIM
TUFLOW FV

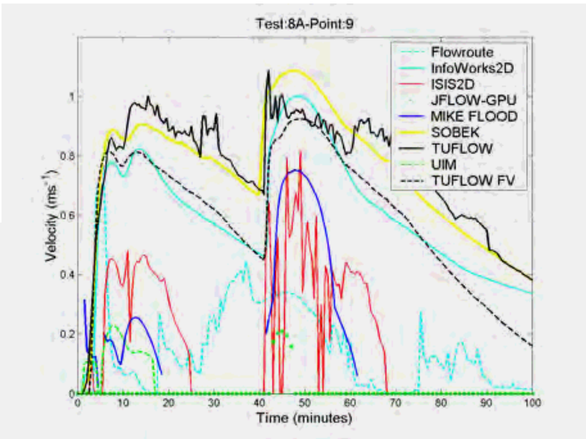

Test:8A-Point:7
Water level (m)
Time (minutes)
FloodRiskMapper
Flowroute
InfoWorks2D
ISIS2D
JFLOW-GPU
MIKE FLOOD
Dynamic RFSM
SOBEK
TUFLOW
UIM
Direct RFSM
TUFLOW FV











Topography:

Local variability in DEM elevations (provided at a 0.5m resolution) meant that discrepancies between models in dry ground elevations (visible on the graphs as the initial level for each curve) were observed due to the various ways in which these elevations are coarsened to the 2m grid resolution (resulting in differences typically within a 0.05-0.1m range, although up to ~0.2m at Point 9[34]). Such differences can affect predictions significantly in models of shallow urban flooding. Deep flow extending over a large number of cells is less affected because these discrepancies are local and their magnitudes and direction at neighbouring cells are not usually correlated[35]. When important flow pathways are governed by features in

[34] A larger ~0.25m discrepancy is observed at Point 6 in the dry ground elevation returned by the RFSM, due to the fact that the RFSM simulation was based on a DEM coarsened from 0.5m to 10m.

[35] Although this will also depend on the rule used in the coarsening of the DEM to the grid resolution (minimum, maximum, average, mid-point, etc.)

the topography that have dimensions comparable to or smaller than the grid resolution the effects of these model grid discrepancies downstream can also be large.

In test 8A (and 8B), these topography effects seem to have been significant at Points 6,8,9 (the depths predicted are of a magnitude similar to the topography errors) and model results at these locations are therefore not commented on any further. These observations however demonstrate that the choice of an adequate grid resolution (finer than 2m in a case similar to the present one) is crucial.

Full models

Water levels

Almost all predictions by the full models at the other points exhibited a 'double-peaked' shape due to the very intense rainfall occurring between t=1min and t=4min, and the point inflow peaking at t~38min. Due to the short travel times the timings of these peaks are mostly in agreement between models, within a few minutes. However at Point 3 (downstream pond) large time lags (up to ~20-25min) in relation to the travel times of typically ~30mins are observed as the levels are rising when the pond is fed by residual shallow flows (especially those due to the rainfall event). Other observations are:

At Point 1 peak depths over 0.5m were predicted. All full models agree in the prediction of this within a range smaller than ~5% of the depth.

At Points 2, 4, 7 maximum depths did not exceed ~0.35m. However all full models agreed in the prediction of the peak levels within ~0.04m at most (this range was also usually smaller in the case of the second peak).

The final levels predicted at Point 3 (downstream pond) were all within a ~0.08m range for a ~0.8m depth (although MIKE FLOOD predicted that the level was still rising at t=300min).

The ISIS results show significant oscillations in areas of shallow flow (mainly 6, 8, 9).

Velocities

Velocity predictions showed variation discrepancies amongst the full models within a range typically up to ~100% (in the peak values predicted). This affected particularly areas of shallow flow (Points 6, 8, 9) where topography effects were significant, or locations where an initial short lived transient peak was observed (1 and 5). The differences were less at locations 2, 3, 4, 7 (within a ~20% range, with the exception of TUFLOW at point 4).

These variations can be explained by a combination of factors: topography effects, differences in the treatment of very shallow flows, differences in the modelling of direct rainfall, differences in the treatment of critical transitions (although the results do not allow a clear distinction in the results between models that do and model that do not possess shock capturing properties).

Simplified models

Water levels

Notes: these comments do not concern points 6,8,9 where large differences are due to topography effects.

Flood Risk Mapper: Predicted water levels were within or close to the range of predictions by the full models, with however some significant differences at some points (e.g. Point 2, with differences of ~30%), and an oscillating solution at some points (e.g. point 3).

Flowroute and JFLOW-GPU: Predicted water levels were usually close to or within the range of predictions by the full models, with however some more significant differences at some points (e.g. Point 2, with differences of ~20%)

RFSM Dynamic: Relatively large differences (from the full model predictions) of magnitude often comparable to the depths, at most points. These are explained by the simplified nature of the solver.

UIM predicted a few peak levels in disagreement with the full models, by up to 0.1m, which represented a significant proportion of the depth (i.e. up to ~30% at Point 4).

Velocities

JFLOW-GPU: Velocity predictions were within the range of predictions by the full models at at some locations (e.g. 2, 4, 7), but otherwise they oscillated.significantly.

Flowroute: Most velocity predictions were well outwith the range of predictions by the full models (with some exceptions, e.g. Point 4) or oscillatory.

Flood Risk Mapper and RFSM Dynamic: no velocity outputs were provided.

UIM: Most velocity predictions were well outwith the range of predictions by the full models (with some exceptions, e.g. Points 2 and 6).

4.7.10 Output in raster format

Observations consistent with the comments above can be obtained from Figure 20, including: 1) Significantly larger extent of inundation according to ISIS (e.g. around points 6 and 8, due to the oscillating solution making the extent covered by the 20cm contour line much larger); 2) significantly larger inundation extent predicted by the RFSM Dynamic (e.g. around Point 1, consistently with the higher level predicted); 3) to a lesser degree, larger inundation extent at Point 1 predicted by UIM; 4) General disagreement between most models at locations where the flow was shallow, e.g. Point 2. However models all agree in the prediction of the outline of the large pond where points 3 and 5 lie, although with differences reaching ~10m in the location of the contour.

4.7.11 Conclusions from test 8A

Most **full models** predicted similar results in terms of peak water levels within a range of a few centimetres. Such differences are unlikely to be larger than the required accuracy of predictions at locations where depths are several times larger, but in a practical problem may affect flow predictions as urban flooding is often shallow.

The topographic effects observed suggest that a 2m grid is insufficiently fine for high-resolution urban flood modelling.

The differences observed in the velocity predictions by the full models (up to ~100%) suggest that predictions of hazard by any particular model are unlikely to be consistently predicted using a 2m resolution grid.

The water level predictions by ISIS were oscillated at some locations where the flow was predicted to be shallow.

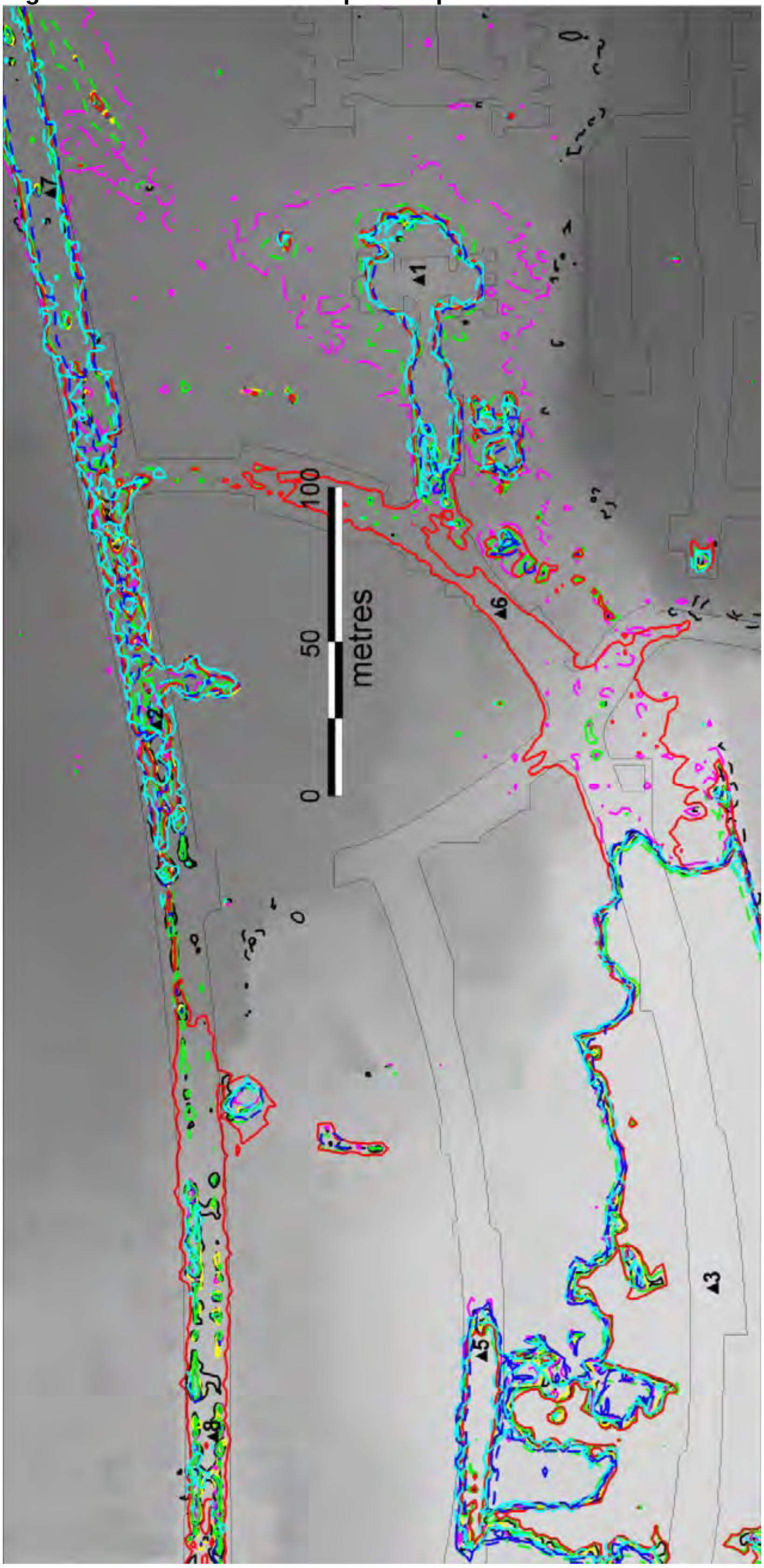
There were considerable discrepancies in the predictions of travel times due to rainfall flooding (up to ~100%).

The **simplified models** Flood Risk Mapper, Flowroute, JFLOW-GPU and UIM made predictions of water levels and arrival times similar to those of the full models, with however some differences in depths which may be excessive for this type of application. These models however made significantly less robust predictions of velocities, likely to be inaccurate for this type of application.

The RFSM Dynamic predicted significantly different arrival times and peak levels that are unlikely to be accurate, with errors significant in most practical applications of this type. Velocity predictions by RFSM are at the present time not available.

The RFSM Direct is inadequate for this type of application as it predicts only a final state.

Figure 20: 20cm contours of peak depth. Colours consistent with other figures.



4.7.12 Summary of relevant technical information

TEST 8A (1) Name	(2) Version (+ numeric. scheme)	(3) Hardware	(4) Multi-Proc.	(5) Grid (2m or 97000 elements)	(6) Time-stepping	(7) Run time (min)
ANUGA	*Not tested*					
Flood Risk Mapper	FRM 0.26	Intel® Core™ Duo, T2500 @ 2.00Ghz, 1GB of RAM	Yes 2 proc.	2.5m	Adaptive	184
FloodFlow	W.12.0 Beta ADI	Intel(R) Core™ 2 CPU 6400 2.13GHz RAM 2GB	no	2m	Adaptive	4
Flowroute	2.9.8	2.4Ghz (Intel Q6600) RAM 4GB	OMP	2m	0.01s	126
InfoWorks	RS 2D v10.5	Intel® Core™ i7 920 2.66GHz Quad Core 12 GB RAM (DDR3-1333)	OMP	101959 triangles	missing	27.1
ISIS	3.3 ADI	Quad Intel Xeon DP 5050 @ 3.0 GHz, 4096MB RAM (FB-DDR2)	Partial, see section 4.0.3	2m	0.25s	78.7
JFLOW-GPU	JFLOW-GPU DW	AMD Phenom II X4 940 3.0 GHz RAM 2.25 GB GPU: NVIDIA GeForce GTX 295	Yes - GPU	2m	Avg: 0.0094s	16.2
MIKE FLOOD	2009 incl. Serv. Pack 2	Intel Core 2 Quad CPU Q9450 2.66 GHz RAM 3.48 GB	no	2m	1s	12.6
RFSM (Direct)	3.5.4	3.5.4 Intel Dual Xeon 2 cores of 3GHz RAM 2GB	No	1111 IZs Based on a 10m grid	N/A	<1s
RFSM (Dynamic)	0.1	3.5.4 Intel Dual Xeon 2 cores of 3GHz RAM 2GB	No	1111 IZs Based on a 10m grid	5s	23.3
SOBEK	2.13	Intel i7 8 core CPU 2.66 GHz	no	2m	10s	24.9
TUFLOW	2010-01-AD-iDP	Intel Core 2 Duo T9800 2.93GHz RAM 4Gb	no	2m	1s	34.2
TUFLOW FV	*2nd order solution not tested*	Intel Xeon X5472 3.00GHz RAM 8Gb	Yes – 8 CPUs	2m	Average time step: 0.01s	72.6 (13.9)
UIM	2009.10	Dual Quad-core 2.83GHz Intel Xeon E5440 Harpertown node RAM 16GB	OMP	2m	0.05s	307.8

Reasons for undertaking Test 8A at a different grid resolution:

Flood Risk Mapper:

“FRM does not have the capability to run at 2m”

Heriot Watt University comment: surprising given that the resolution of the DEM was 0.5m.

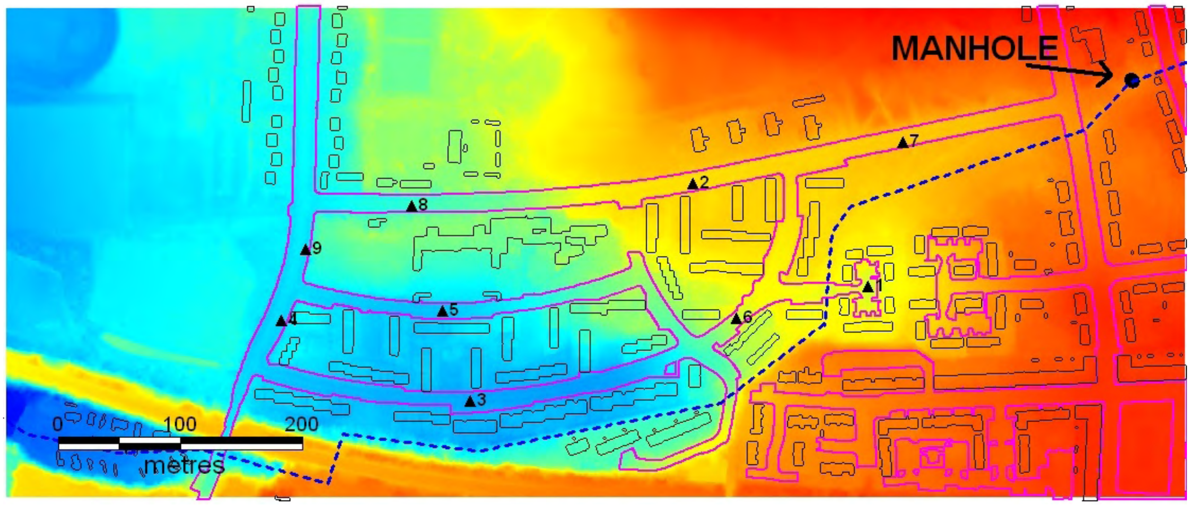
4.8 Test 8B: Surface flow from a surcharging sewer in urban areas

4.8.1 Introduction

This test (see Appendix A.8 for details) is based on the same site, DEM and modelled area as in test 8A. A culverted watercourse of circular section is assumed to run through the site, with a single manhole at the location indicated in Figure 21. An inflow boundary condition is applied at the upstream end of the pipe, with a surcharge expected to occur at the manhole. The flow from the above surcharge spreads across the surface of the DEM.

Participants were expected to take into account the presence of a large number of buildings in the modelled area, and apply a land-cover dependent roughness value, with 2 categories: 1) Roads and pavements; 2) Any other landcover.

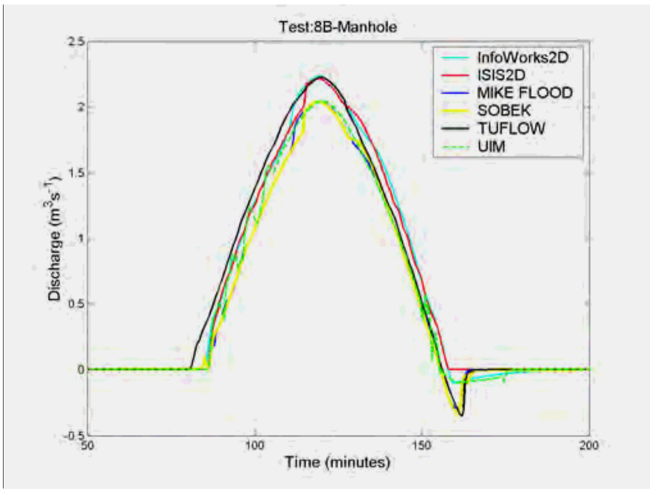
Figure 21: DEM used, with the location of the manhole. The course of the pipe is irrelevant to the modelling. Triangles: output point locations.



This tests the package's capability to simulate shallow inundation originating from a surcharging underground pipe, at relatively high resolution (2m). The pipe is modelled in 1D and connected to the 2D grid through the manhole.

4.8.2 Output as time series

4.8.2.1 Manhole discharge

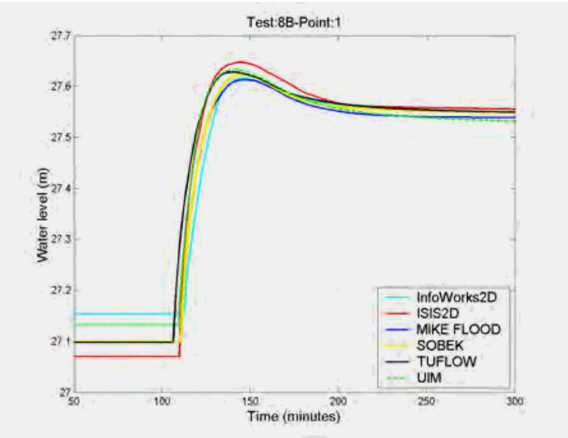


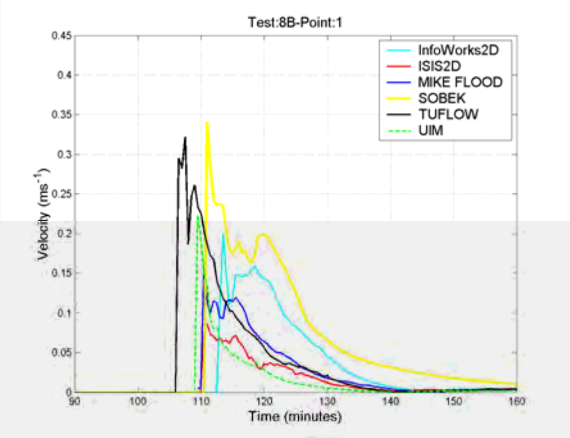
The manhole flow predictions were generally very similar to each other, although with total volumes differing within a ~15% range, as shown the table below.

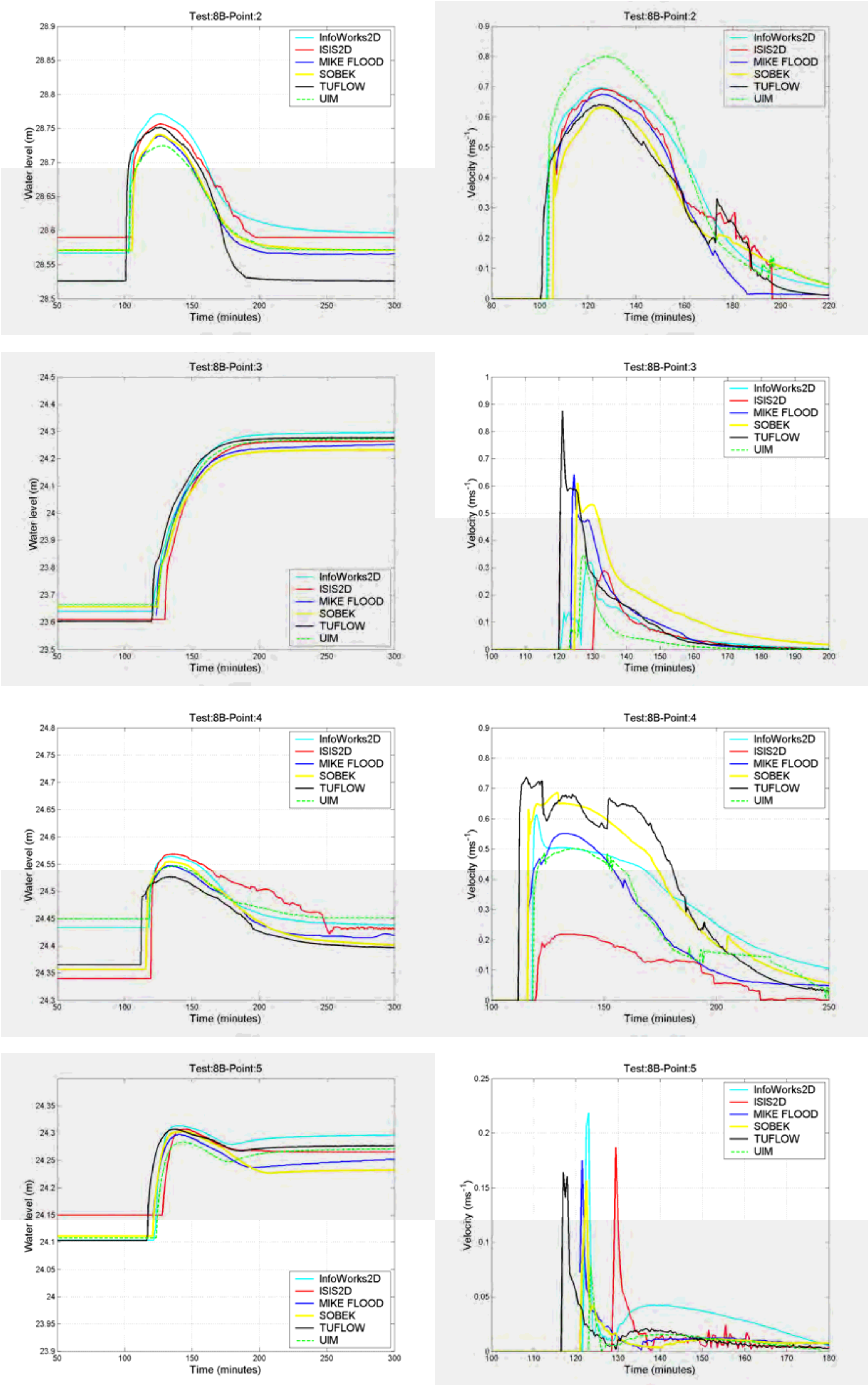
Package	Volume (m^3)
Infoworks	5873.1
ISIS	5864.5
TUFLOW	5837.4
UIM	5226.8
MIKE FLOOD	5024.4
SOBEK	4987.0

4.8.2.2 Water levels and velocities

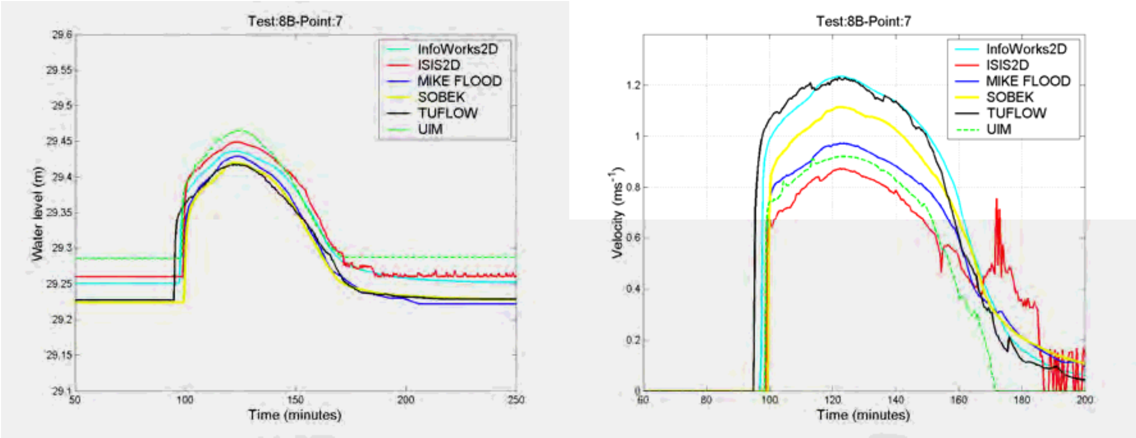
Results from points 6, 8, 9 are not shown because of ‘topographic’ effects similar to those observed in Test 8A.







Test:8B-Point:2
Water level (m)
Time (minutes)
Velocity (ms⁻¹)
Test:8B-Point:3
Test:8B-Point:4
Test:8B-Point:5
InfoWorks2D
ISIS2D
MIKE FLOOD
SOBEK
TUFLOW
UIM



Water levels

Very similar comments to those made for the test 8A results can be made, as follows:

At Point 1 peak depths over 0.5m were predicted. All models agree in the prediction of this within a range smaller than ~6% of the depth.

At Points 2,4,7 maximum depths did not exceed ~0.20m. However all full models agreed in the prediction of the peak levels within ~0.04m. UIM predicted a peak level within or slightly outwith this range.

The final levels predicted at Point 3 (downstream pond) were all within a ~0.08m range for a ~0.7m depth at Point 3.

Velocities

There are considerably larger differences in the predictions of velocities, up to 50% or more (e.g. Point 4).

Some of the differences observed are due to differences (of total volume or of peak discharge) in the prediction of the outflow from the manhole (e.g. the peak velocities at point 7 are correlated with the peak discharges from the manhole). However most of the differences observed are explained as in Test 8A, i.e.: 1) topography effects due to the insufficient 2m resolution; 2) differences in the modelling of shallow flows; 3) differences in the treatment of critical transitions (shocks).

4.8.3 Conclusions from test 8B

The packages taking part in Test 8B have demonstrated their ability to link a 1D pipe model to a 2D overland flow through a manhole. This functionality is however not supported in the current versions of Flood Risk Mapper, Flowroute, JFLOW-GPU, RFSM, TUFLOW FV or ANUGA.

Conclusions similar to those in Test 8B can otherwise be made regarding the accuracy of the 2D flow predictions, see Section 4.7.11.

4.8.4 Summary of relevant technical information

TEST 8B (1) Name	(2) Version (+ numeric. scheme)	(3) Hardware	(4) Multi-Proc.	(5) Grid (2m or 97000 elements)	(6) Time-stepping	(7) Run time (min)
ANUGA	*Not tested*					
Flood Risk Mapper	*Not tested*					
FloodFlow	W.12.0 Beta ADI	Intel(R) Core™ 2 CPU 6400 2.13GHz RAM 2GB	No	2m	Adaptive	6
Flowroute	*Not tested*					
InfoWorks	RS 2D v10.5	Intel® Core™ i7 920 2.66GHz Quad Core 12 GB RAM (DDR3-1333)	No	94815 triangles	Adaptive	6.0
ISIS	3.3 TVD	Quad Intel Xeon DP 5050 @ 3.0 GHz, 4096MB RAM (FB-DDR2)	No	2m	0.05s	734.3
JFLOW-GPU	*Not tested*					
MIKE FLOOD	2009 incl. Serv. Pack 2	Intel Core 2 Quad CPU Q9450 2.66 GHz RAM 3.48 GB	No	2m	1s	2
RFSM Dyn. or dir	*Not tested*					
SOBEK	2.13	Intel i7 8 core CPU 2.66 GHz	No	2m	5s	18.9
TUFLOW	2010-01-AD-iDP	Intel Core 2 Duo T9800 2.93GHz RAM 4Gb	No	2m	1s	9.2
TUFLOW FV	*Not tested*					
UIM	2009.10	Intel Core™ 2 Duo CPU T7800 2.60GHz RAM 3GB	No	2m	Adaptive 0.039s - 10s Avg 0.055s	743.3

Other information provided:

ISIS: "... for test case 8B, we used the TVD Solver. The reason for this is the likelihood of extensive supercritical flows in simulating test case 8B."

SOBEK: The top of the manhole as instructed, lies at elevation 31.46 m. This is "in a kind of pond (or area surrounding by higher grounds). The water level in this pond has to rise above 31.86 m, before the water flowing out off the surcharging culvert can flow out of this pond and inundate other parts of the modelled 2D landscape. Due to this pond, we observed that the actual maximum surcharging-culvert-discharge is smaller than in a situation, where there is not such pond in the 2D landscape.

4.9 Run times

Computional times (reported in Table 5) vary within up to 4 orders of magnitude. This is explained by a) the choice of time step (this is partly imposed by the numerical approach, although simulations may have been run with time steps shorter than necessary); b) the number of iterations performed at each time steps; c) the efficiency of the numerical algorithm; d) the use (or not) of multi-processing; e) any ‘overhead’ computational costs and f) hardware specification. The information available to the authors does not allow to fully explain the discrepancies observed however, the reported times are not considered to be excessive for practical flood risk management applications.

Table 5: Summary of run times for all tests and packages

	Test 1 (s)	Test 2 (min)	Test 3 (s)	Test 4 (min)	Test 5 (min)	Test 6A (min)	Test 6B (min)	Test 7 (min)	Test 8A (min)	Test 8B (min)
ANUGA	205.00	18.80	6.00	60.80	69.30	11.50	23.10			
Flood Risk Mapper		18.00	10.00	27.00					184.00	
FloodFlow	300.00	130.00		75.00	350.00			50.00	4.00	6.00
Flowroute	240.00	24.00	74.00	92.00	112.00				126.00	
InfoWorks	16.00	0.73	10.00	6.50	0.70	1.30	2.60	87.00	27.10	6.00
ISIS	48.00	1.58	23.00	28.70	47.00	362.00	1186.30	51.00	78.70	734.30
JFLOW-GPU		1.83	27.40	2.30	10.20		110.50		16.20	
MIKE FLOOD	3.00	0.40	1.00	1.27	0.68	1.29	1.45	11.27	6.38	2.08
RFSM (Direct)		0.02	1.00	0.02	0.02				0.02	
RFSM (Dynamic)	72.00	0.19		5.80	9.80				23.30	
SOBEK	17.00	1.67	20.00	16.90	2.80	6.50	16.90	194.90	24.90	18.90
TUFLOW	16.60	1.92	1.80	5.10	0.60	2.60	8.70	9.90	34.20	9.20
TUFLOW FV 1st order	6.40	0.60	0.94	5.00	1.40	1.30	2.80		13.90	
TUFLOW FV 2nd order	25.60	2.64	2.93	24.50	2.90	7.10	16.10		72.60	
UIM	349.00	60.05	56.00	282.80	44.50				307.80	743.30

5. Conclusions

5.1 Conclusions for packages based on the shallow water equations.

As can be seen from Table 6 the shallow water equation packages (ANUGA, InfoWorks 2D, ISIS 2D, MIKE FLOOD, SOBEK, TUFLOW, TUFLOW FV) have been applied to all of the test cases. The detailed discussion of the predictions from each test in Section 4 leads to the general conclusion that the packages have produced comparable predictions of water level and velocity across the full range of tests and are applicable across the full range of Environment Agency flood risk modelling requirements.

Caveats to this general conclusion are:

1. For large scale valley flooding due to dam break (Test 5), predictions obtained by TUFLOW (FD), ANUGA and to a lesser extent MIKE FLOOD (FD) oscillate in some locations. This results in higher peak water level and velocity predictions than those obtained by the other shallow water equation packages. For MIKE FLOOD and TUFLOW this is likely to be due to the lack of shock capturing capability of their numerical scheme. The reason that the ANUGA results exhibit this behaviour requires further investigation. Using predictions that contain oscillations to create contour maps may result in higher values for extremes compared with solutions that do not contain oscillations. This could be significant in mapping maximum flood hazard and maximum flood outline.
2. For predictions of dam break at the laboratory scale (Test 6A), there is reasonable agreement between predictions and measured water levels for all the packages but significant differences in predictions of velocities compared to laboratory measurements. The codes based on shock capturing numerical schemes perform better overall in terms of both water level and velocity prediction. When applied to the dam break simulation at the field scale (Test 6B) the variation in predictions is less significant. For such simulations it is therefore prudent to use packages that employ a shock capturing numerical scheme.
3. For 1D river to 2D floodplain linking (Test 7), close agreement is obtained for water level and velocity predictions in the river channel however, significant variations in water level and velocity occur for the comparisons on all three floodplains. These differences are also reflected in the contour plots of peak velocities. These are due to variations in the way each package predicts the flood volume exchanged between the main channel and the floodplain, for which there is no consistent approach used in practice at the present time. This calculation is also very sensitive to the representation of river bank levels in the calculation of volume exchange. Further research into this aspect of model linking is required.
4. For rainfall and point source generated surface flow in urban areas (Test 8A), the manner in which the underlying floodplain topography is represented has a significant influence on water level and velocity predictions. Improvements in the consistency of predictions between packages could be achieved by using a higher resolution grid than adopted here, although this approach would also increase model run times.

A direct comparison of the computational efficiency of each package is not possible due to differences in the hardware used and the time increments selected by the modeller. However, the run times reported are considered acceptable for use on Environment Agency applications at the scale tested here. However, their computational requirements make their use for regional and national flood risk assessment over large areas and for probability / uncertainty analysis requiring multiple runs with varying parameter values impractical.

It is possible to obtain predictions over large areas by using shallow water equation models with a very course grid resolution however, the averaging of parameters necessary to achieve this means the quality of the predictions obtained are unlikely to be better than those one would get from a simplified modelling approach.

Current research in uncertainty estimation is investigating the use of parallel computing techniques to obtain the multiple runs necessary to support the probability analysis required for this application.

5.2 Conclusions for packages based on simplified equations

In general, water levels predicted by software based on simplified equations (Flowroute, JFLOW-GPU and UIM) predict water levels comparable to those obtained from shallow wave equation simulations. Consequently, packages based on simplified equations are suitable to support decision making where predictions of dynamic inundation extent and/or maximum depth are required, e.g. catchment flood management planning and flood risk assessment.

Where their performance is less comparable to shallow water equation models is in the prediction of velocities (with solutions often oscillatory in highly unsteady flow conditions), and in areas where momentum conservation is important, such as the prediction of water levels and velocities in the complex flow field close to a dam break (Test 5) and where the spreading flood encounters an adverse slope on the floodplain (Test 3). It is recommended that this class of model is not used for flood hazard mapping where velocity predictions are required. For applications such as strategic flood risk assessments, contingency planning and reservoir inundation mapping consideration should be given as to whether accurate velocity predictions are important, if so then packages based on simplified equations should not be used.

For the tests undertaken in this study runtimes for shallow water equation and simplified models are often comparable. Based on this evidence there is no benefit in reduced computational effort in applying simplified models compared to shallow water equation models. However, this maybe a consequence of the scale of the tests used here which are over smaller domains than one would typically apply a simplified model to. Additionally, this report does not consider model set up time and ease of application, which was previously reviewed in Environment Agency (2009).

5.3 Conclusions for packages based on volume spreading

RFSM (Direct) produces predictions of final inundation extent and depth that compare well with shallow water equation models, for the limited number of tests to which it was applied, namely where dynamic effects are insignificant in determining water movement. This limits its applicability to large scale application where its short run time provides a practical benefit, i.e. national scale probabilistic flood risk assessment and broad scale catchment flood management planning.

Table 6: Suitable Packages for Environment Agency applications.

Application	Predictions required	Suitable packages identified in study
National scale probabilistic flood risk assessment (e.g. NaFRA)	i. inundation extent	Usable predictions could be obtained using all packages discussed above, although computational efficiency of RFSM (Direct) is a significant advantage.
Strategic / broad-scale flood risk assessments, rapid reservoir inundation mapping and contingency planning for real time flood risk management	i. inundation extent ii. maximum depth	Appropriate predictions will be obtained using packages based on the shallow water equations or simplified equations. The need for detail mitigates against the use of RFSM (Direct).
Detailed flood hazard assessments, detailed reservoir inundation mapping and site-specific FRAs	i. inundation extent ii. maximum depth iii. maximum velocity	The most suitable packages for this task are those based on the shallow water equations. If sub-critical to super-critical or super-critical to sub-critical flow transitions exist there may be benefit in using codes based on shock capturing numerical schemes, see Table 3

5.4 Conclusions for other packages

FloodFlow predictions were undertaken using a Manning's n value that varied with depth rather than the constant value specified in the benchmarks. As a result the predictions provided (reported here in Appendix D) are not comparable with those from the other packages. It is therefore only possible to provide the qualitative observation that the predictions are consistent with what one would expect from applying a depth varying Manning's n to these tests.

The quality of predictions made by Flood Risk Mapper and RFSM (Dynamic) show considerable variation with those from shallow water equation models for water level predictions (except in Tests 2 and 4) and velocities are not predicted at this stage. RFSM (Dynamic) remains the subject of future research.

5.5 Future Use of Benchmark Data and Results

For further information on accessing the benchmarking data and the results from the study please contact: fcerm.evidence@environment-agency.gov.uk.

5.6 Data review and updating

As reliance on 2D flood inundation model predictions is likely to grow it is recommended that the Environment Agency consider reviewing the suitability of the benchmark tests in the future to identify the need to update these to take account of modelling trends. The principle area where further work is likely to be required is in model linking. This is an area where significantly more research might be undertaken to ensure consistency of model prediction for Environment Agency application.

5.7 Future applicability

The results and conclusions in this report are accurate at the time of publication, but they represent a 'snap-shot' in time. It is likely that development work will be undertaken on the software packages discussed in this report, and so in time it is possible that the results and conclusions may become less relevant to individual software packages. However, the conclusions which compare the generic use of models using the full equations and those using the simplified equations will probably be relevant over a longer time period.

6. References

Environment Agency, 2009. Desktop review of 2D hydraulic modelling packages. Environment Agency Science Report-SC080035/SR.

SOARES-FRAZAO, S. AND ZECH, Y., 2002 Dam-break flow experiment: The isolated building test case. Available online at: http://www.impact-project.net/wp3_technical.htm

APPENDIX A – Test specifications

Test 1 – Flooding a disconnected water body

1. Modelling performance tested

The objective of the test is to assess basic package capabilities such as handling disconnected water bodies and wetting and drying of floodplains.

2. Description

This test consists of a sloping topography with a depression as illustrated in Figure (a). The modelled domain is a perfect 700m x 100m rectangle. A varying water level, see Figure (b), is applied as a boundary condition along the entire length of the left-hand side of the rectangle, causing the water to rise to level 10.35m. This elevation is maintained for long enough for the water to fill the depression and become horizontal over the entire domain. It is then lowered back to its initial state, causing the water level in the pond to become horizontal at the same elevation as the sill, 10.25m.

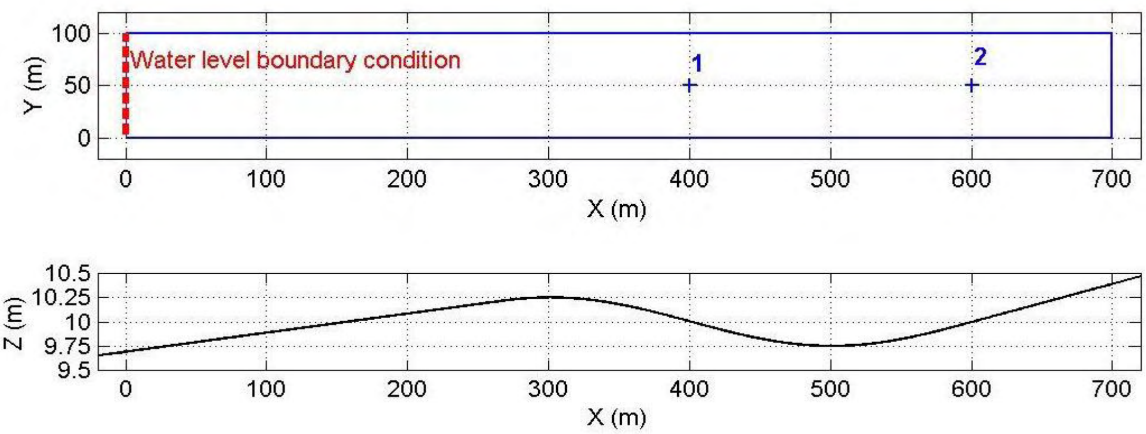


Figure (a): Plan and profile of the DEM use in Test 1. The area modelled is a perfect rectangle extending from X=0 to X=700m and from Y=0 to Y=100m as represented.

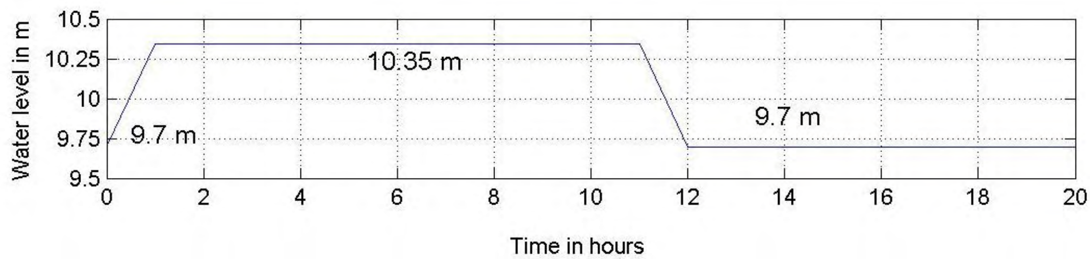


Figure (b): Water level hydrograph used as boundary condition (table provided as part of dataset).

3. Boundary and initial conditions

Varying water level along the dashed red line in Figure (a). Table provided as part of dataset.
All other boundaries are closed.
Initial condition: Water level elevation = 9.7m.

4. Parameter values

Manning's n: 0.03 (uniform)

Model grid resolution: 10m (or 700 nodes in the area modelled)

Time of end: the model is to be run until time t = 20 hours

5. Required output

Software package used: version and numerical scheme.

Specification of hardware used to undertake the simulation: processor type and speed, RAM.

Minimum recommended hardware specification for a simulation of this type.

Time increment used, grid resolution (or number of nodes in area modelled) and total simulation time to specified time of end.

Water level versus time (output frequency 60s), at two locations in the pond as shown in Figure (a) and provided as part of the dataset.

6. Dataset content

Description	File Name
Georeferenced Raster ASCII DEM at resolution 2m	Test1DEM.asc
Upstream boundary condition table (water level vs. time)	Test1BC.csv
Location of output points	Test1Output.csv

7. Additional comments

Participants are asked to provide model results **at least** for the grid resolution specified above.

Model results for 1 alternative resolution or mesh may also be provided.

Participants are asked to justify their reasons for not carrying out the test, or for carrying out the test using an alternative resolution.

Test 2 – Filling of floodplain depressions

1. Modelling performance tested

The test has been designed to evaluate the capability of a package to determine inundation extent and final flood depth, in a case involving low momentum flow over a complex topography.

2. Description

The area modelled, shown in Figure (a), is a perfect 2000 m x 2000 m square and consists of a 4 x 4 matrix of ~0.5 m deep depressions with smooth topographic transitions. The DEM was obtained by multiplying sinusoids in the North to South and West to East directions and the depressions are all identical in shape. An underlying average slope of 1 : 1500 exists in the North to South direction, and of 1 : 3000 in the West to East direction, with a ~2m drop in elevation along the North-West to South-East diagonal.

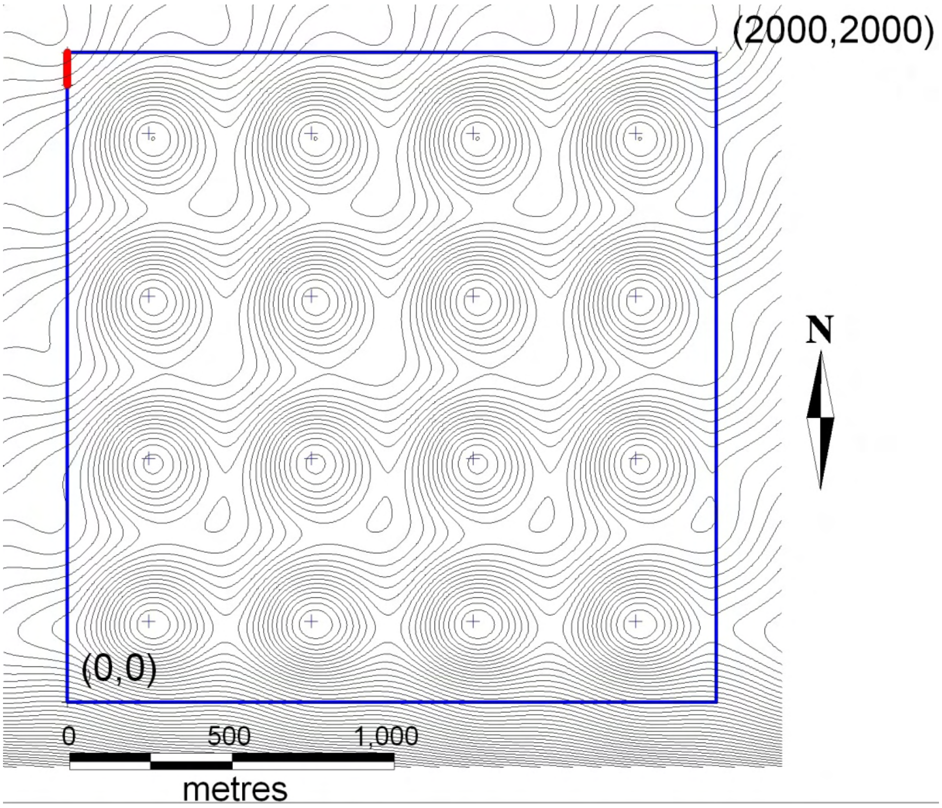


Figure (a): Map of the DEM showing the location of the upstream boundary condition (red line), ground elevation contour lines every 0.05 m, and output point locations (crosses).

The inflow boundary condition is applied along a 100m line running South from the North Western corner of the modelled domain, see Figure (a).

A flood hydrograph with a peak flow of 20m^3/s and time base of ~85mins is used. The model is run for 2 days (48 hours) to allow the inundation to settle to its final state.

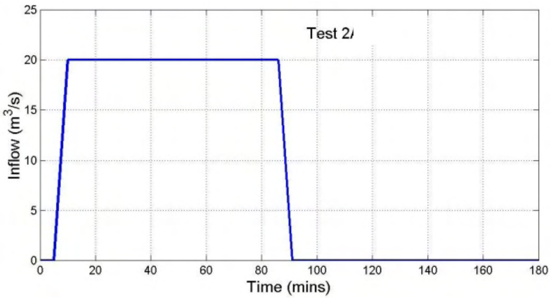


Figure (b): Inflow hydrograph used as upstream boundary condition in Test 2.

3. Boundary and initial conditions

Inflow along the red line in Figure (a). Location and tables provided as part of dataset.

All other boundaries are closed.

Initial condition: Dry bed.

4. Parameter values

Manning's n: 0.03 (uniform)

Model grid resolution: 20m

(or ~10000 nodes in the area modelled)

Time of end: model is to be run until time t = 48 hours

5. Required output

Software package used: version and numerical scheme.

Specification of hardware used to undertake the simulation: processor type and speed, RAM.

Minimum recommended hardware specification for a simulation of this type.

Time increment used, grid resolution (or number of nodes in area modelled) and total simulation time to specified time of end.

Total water volume on the floodplain at the end of the simulation.

Numerical prediction of **water level** and versus time at the centre of each depression (coordinates provided as part of dataset).

Output frequency: 300s.

6. Dataset content

Description	File Name
Georeferenced Raster ASCII DEM at resolution 2m	Test2DEM.asc
Upstream boundary condition table (inflow vs. time)	Test2_BC.csv
Outline of modelled area (shapefiles)	Test2ActiveArea_region
Location of upstream boundary condition (shapefile)	Test2BC_polyline
Location of output points	Test2Output.csv

7. Additional comments

Linear interpolation should be used to interpolate inflow values.

Participants are asked to provide model results **at least** for the grid resolution specified above.

Model results for one alternative resolution or mesh may also be provided.

Participants are asked to justify their reasons for not carrying out the test, or for carrying out the test using an alternative resolution.

Test 3 – Momentum conservation over a small obstruction.

1. Modelling performance tested

The objective of the test is to assess the package's ability to conserve momentum over an obstruction in the topography. This capability is important when simulating sewer or pluvial flooding in urbanised floodplains. The barrier to flow in the channel is designed to differentiate the performance of packages without inertia terms and 2D hydrodynamic packages with inertia terms. With inertia terms some of the flood water will pass over the obstruction.

2. Description

This test consists of a sloping topography with two depressions separated by an obstruction as illustrated in Figure (a). The dimensions of the domain are 300m longitudinally (X) x 100m transversally (Y). A varying inflow discharge, see Figure (b), is applied as an upstream boundary condition at the left-hand end, causing a flood wave to travel down the 1:200 slope. While the total inflow volume is just sufficient to fill the left-hand side depression at x=150m, some of this volume is expected to overtop the obstruction because of momentum conservation and settle in the depression on the right-hand side at x=250m. The model is run until time T=900s (15 mins) to allow the water to settle.

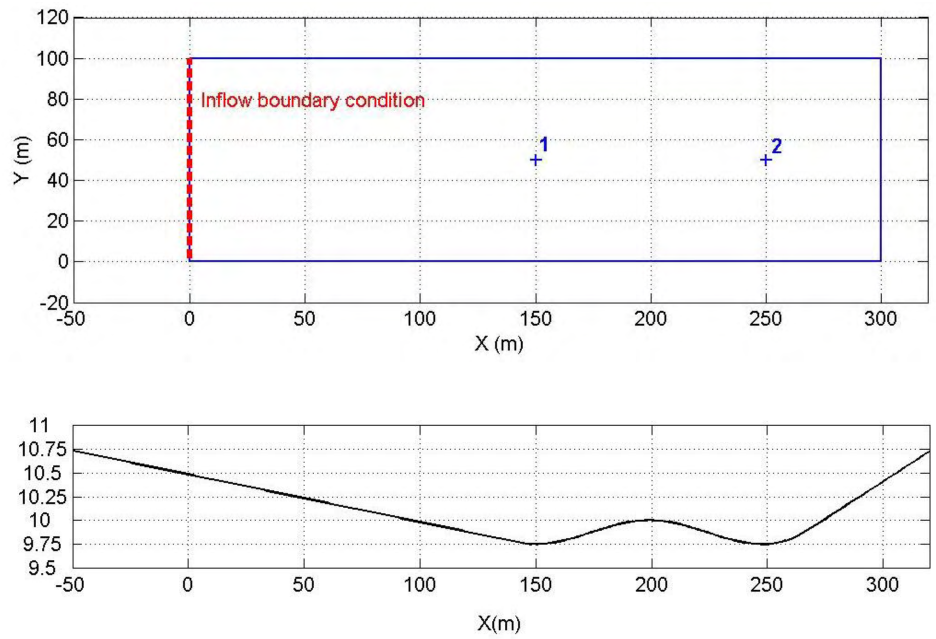


Figure (a): Plan and profile of the DEM use in Test 3. The area modelled is a perfect rectangle extending from X=0 to X=300m and from Y=0 to Y=100m as represented.

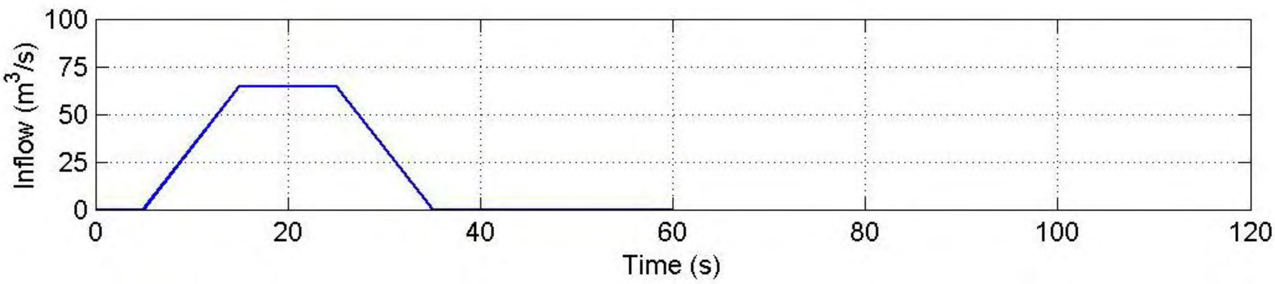


Figure (b): Inflow hydrograph used as upstream boundary condition.

3. Boundary and initial conditions

Inflow boundary condition along the dashed red line in Figure (a). Table provided as part of dataset.

All other boundaries are closed.

Initial condition: Dry bed.

4. Parameter values

Manning's n: 0.01 (uniform)

Model grid resolution: 5m

(or ~1200 nodes in the area modelled)

Time of end: the model is to be run until time t = 15 mins

5. Required output

Software package used: version and numerical scheme.

Specification of hardware used to undertake the simulation: processor type and speed, RAM.

Minimum recommended hardware specification for a simulation of this type.

Time increment used, grid resolution (or number of nodes in area modelled) and total simulation time to specified time of end.

Numerical predictions of **velocity** and **water level** versus time (output frequency 2s) at location 1 (centre of the first depression) defined below.

Numerical predictions of **water level** versus time (output frequency 2s) at location 2 (centre of the second depression) defined below.

Location	X	Y
1	150	50
2	250	50

6. Dataset content

Description	File Name
Georeferenced Raster ASCII DEM at resolution 2m	Test3DEM.asc
Upstream boundary condition table (discharge vs. time)	Test3BC.csv

7. Additional comments

Linear interpolation should be used to interpolate inflow values.

It is pointed out that results may be significantly affected by the effective modelled domain width in case this is not exactly 100m. Participants are reminded to ensure that the effective domain width is 100m (in this test only an alternative is to multiply the inflow discharge by the appropriate ratio if the effective width is not 100m).

Participants are asked to provide model results **at least** for the grid resolution specified above.

Model results for one alternative resolution or mesh may also be provided.

Participants are asked to justify their reasons for not carrying out the test, or for carrying out the test using an alternative resolution.

Test 4 – Speed of flood propagation over an extended floodplain

1. Modelling performance tested

The objective of the test is to assess the package's ability to simulate the celerity of propagation of a flood wave and predict transient velocities and depths at the leading edge of the advancing flood front. It is relevant to fluvial and coastal inundation resulting from breached embankments.

2. Description

This test is designed to simulate the rate of flood wave propagation over a 1000m x 2000m floodplain following a defence failure, Figure (a). The floodplain surface is horizontal, at elevation 0m. One inflow boundary condition will be used, simulating the failure of an embankment by breaching or overtopping, with a peak flow of 20 m^3/s and time base of ~ 6 hours. The boundary condition is applied along a 20m line in the middle of the western side of the floodplain.

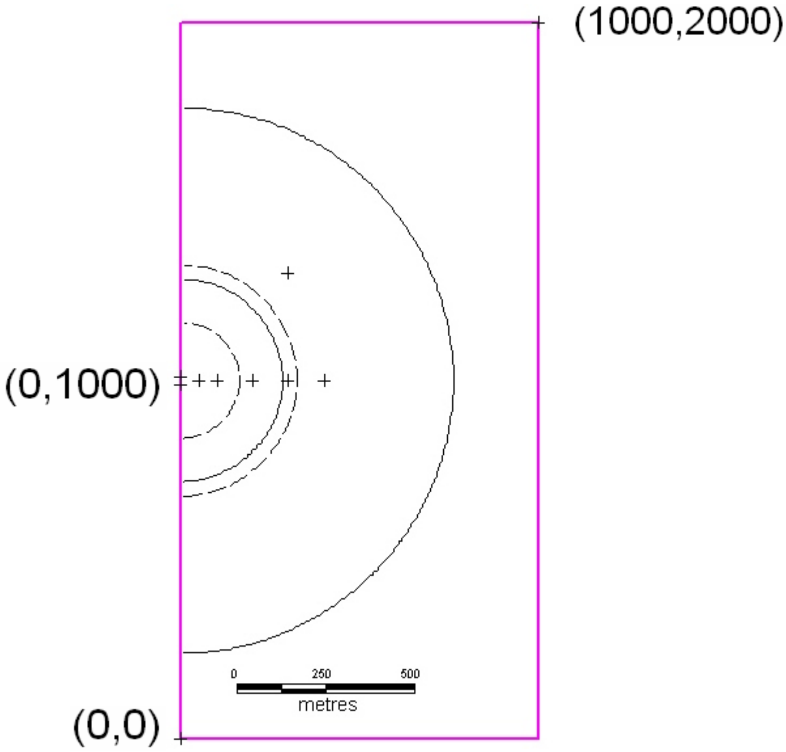


Figure (a): Modelled domain, showing the location of the 20m inflow, 6 output points, and possible 10cm and 20cm contour lines at time 1 hour (dashed) and 3 hours (solid).

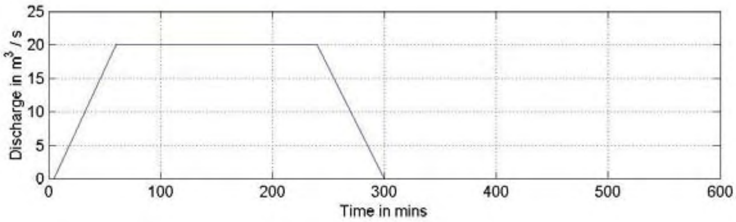


Figure (b): Hydrograph applied as inflow boundary condition

3. Boundary and initial conditions

Inflow boundary condition as shown in Figure (b). Table provided as part of dataset.

All other boundaries are closed.

Initial condition: Dry bed.

4. Parameter values

Manning's n: 0.05 (uniform)

Model grid resolution: 5m

(or ~80000 nodes in the area modelled)

Time of end: the model is to be run until time t = 5 hours (if an alternative end time is used run times must be reported for t=5 hours)

5. Required output

Software package used: version and numerical scheme.

Specification of hardware used to undertake the simulation: processor type and speed, RAM.

Minimum recommended hardware specification for a simulation of this type.

Time increment used, grid resolution (or number of nodes in area modelled) and total simulation time to specified time of end.

Raster grids (or TIN) at the model resolution consisting of:

Depths at times 30mins, 1 hour, 2 hours 3 hours, 4 hours.

Velocities (scalar) at times 30mins, 1 hour, 2 hours 3 hours, 4 hours.

Plots of **velocity** and **water elevation** versus time (suggested output frequency 20s) at the six locations represented in Figure (a) and provided as part of dataset.

6. Dataset content

Description	File Name
Upstream boundary condition table (inflow vs. time)	Test4BC.csv
Location of output points	Test4Output.csv

The model geometry is as specified in Section 2. No DEM is provided as the ground elevation is uniformly 0.

7. Additional comments

Linear interpolation should be used to interpolate inflow values.

Participants are asked to provide model results **at least** for the grid resolution specified above.

Model results for 1 alternative resolution or mesh may also be provided.

Participants are asked to justify their reasons for not carrying out the test, or for carrying out the test using an alternative resolution.

Test 5 – Valley flooding

1. Modelling performance tested

This tests a package's capability to simulate major flood inundation and predict flood hazard arising from dam failure (peak levels, velocities, and travel times).

2. Description

This test is designed to simulate flood wave propagation down a river valley following the failure of a dam. The valley DEM is ~0.8km by ~17km and the valley slopes downstream on a slope of ~0.01 in its upper region, easing to ~0.001 in its lower region. The inflow hydrograph applied as a boundary condition along a ~260m long line at the upstream end is designed to account for a typical failure of a small embankment dam and to ensure that both super-critical and sub-critical flows will occur in different parts of the flow field, see Figure (b). The model is run until time T = 30 hours to allow the flood to settle in the lower parts of the valley.

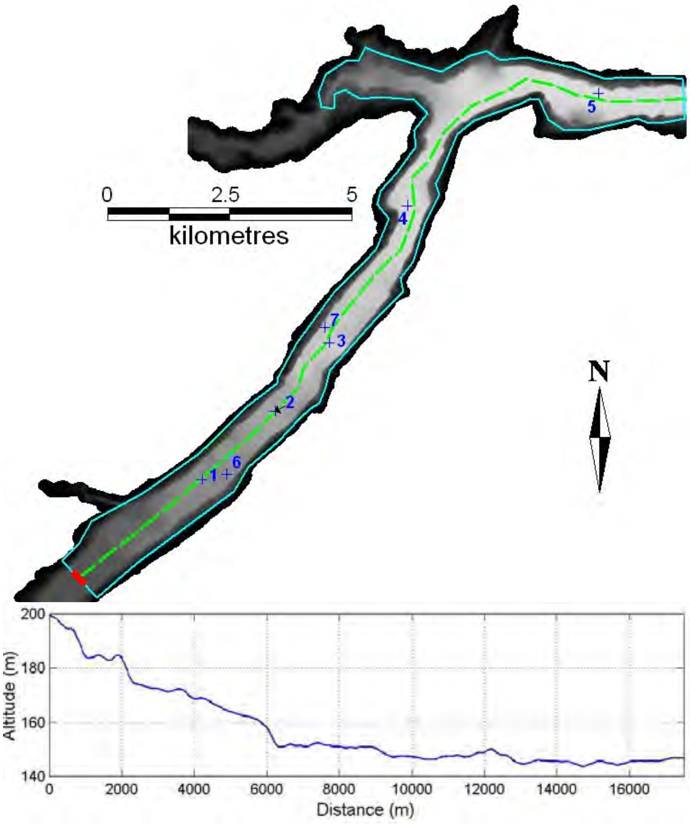


Figure (a): DEM used, with cross-section along the centre line, and location of the output points. The red line indicates the location of the boundary condition and the blue polygon is the modelled area.

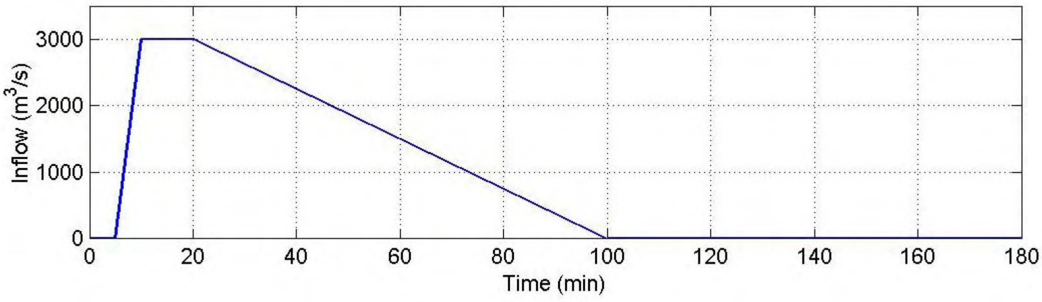


Figure (b): Inflow hydrograph applied in Test 5.

3. Boundary and initial conditions

Inflow boundary condition along the dashed red line in Figure (a). Table provided as part of dataset.

All other boundaries are closed.

Initial condition: Dry bed.

4. Parameter values

Manning's n: 0.04 (uniform)

Model grid resolution: 50m

(or ~7600 nodes in the 19.02 km^2 area modelled)

Time of end: the model is to be run until time t = 30 hours (if an alternative end time is used run times must be reported for t=30 hours)

5. Required output

Software package used: version and numerical scheme.

Specification of hardware used to undertake the simulation: processor type and speed, RAM.

Minimum recommended hardware specification for a simulation of this type.

Time increment used, grid resolution (or number of nodes in area modelled) and total simulation time to specified time of end.

Raster grids (or TIN) at the model resolution consisting of:

a. Peak **water level elevations** reached during the simulation
b. Peak water **depths** reached during the simulation
c. Peak **velocities** (scalar) reached during the simulation

Water level versus time (suggested output frequency 60s), at seven locations as shown in Figure (a) and provided as part of the dataset.

Velocity versus time (suggested output frequency 60s), at seven locations as shown in Figure (a) and provided as part of the dataset.

6. Dataset content

Description	File Name
Georeferenced Raster ASCII DEM at resolution 10m	Test5DEM.asc
Upstream boundary condition table (inflow vs. time)	Test5BC.csv
Outline of modelled area (shapefiles)	Test5ActiveArea_region
Location of upstream boundary condition (shapefile)	Test5BC_polyline
Location of upstream boundary condition (backup text file)	Test5BC_backup.txt
Location of output points	Test5Output.csv

7. Additional comments

The test can be set-up without access to the 2 shapefiles provided in case participants are unable to use these.

Linear interpolation should be used to interpolate inflow values.

Participants are asked to provide model results **at least** for the grid resolution specified above.

Model results for one alternative resolution or mesh may also be provided.

Participants are asked to justify their reasons for not carrying out the test, or for carrying out the test using an alternative resolution.

Tests 6A and 6B – Dam break

1. Modelling performance tested

This tests the capability of each package to correctly simulate hydraulic jumps and wake zones behind buildings using high-resolution modelling.

2. Description

This dam-break test case has been adapted from an original benchmark test case available from the IMPACT project (IMPACT, 2004; Soares-Frazao and Zech, 2002), for which measurements from a physical model at the Civil Engineering Laboratory of the Université Catholique de Louvain (UCL) are available.

Test 6A is the original test proposed in Soares-Frazao and Zech 2002, where the physical dimensions are those of the laboratory model. The test involves a simple topography, a dam with a 1m wide opening, and an idealised representation of a single building downstream of the dam, see Figure (a). An initial condition is applied, consisting in a uniform depth of 0.4m upstream from the dam, and 0.02m downstream from the dam. The flow is contained by vertical walls at all boundaries of the DEM.

Test 6B is identical to Test 6A although all physical dimensions have been multiplied by 20 to reflect realistic dimensions encountered in practical flood inundation modelling applications.

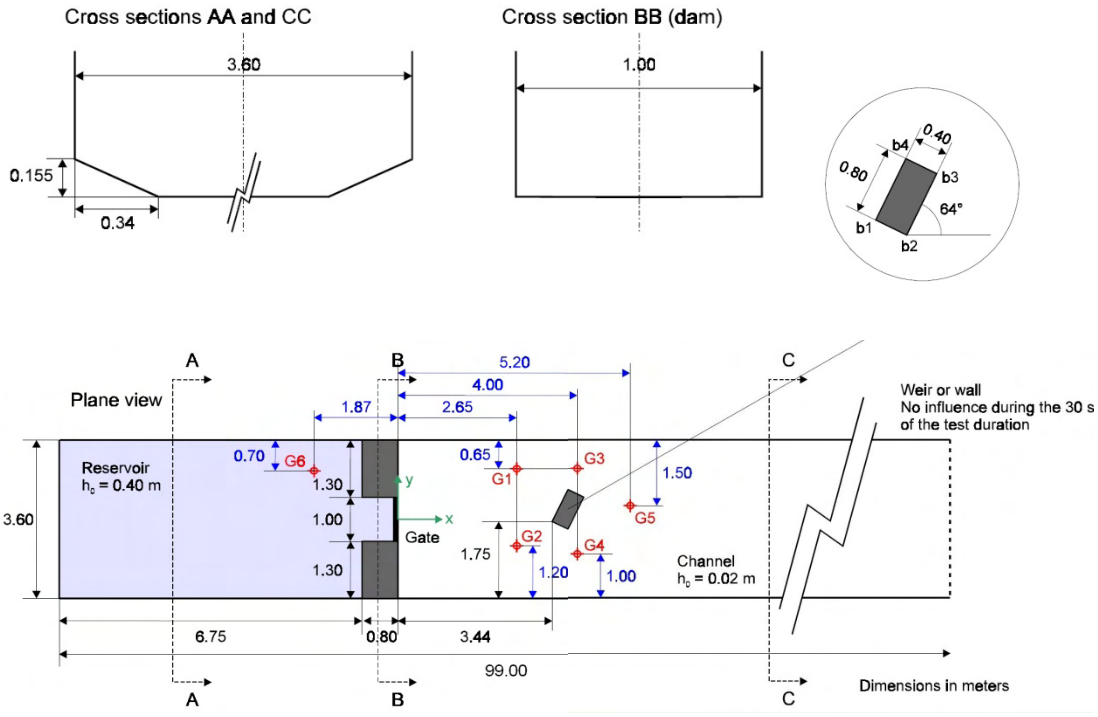


Figure (a): Set-up for Test 6A (adapted from Soares-Frazao and Zech, 2002).

3. Boundary and initial conditions

No boundary condition specified as the flow is contained by vertical walls.

Initial condition:

In Test 6A: Depth = 0.4m upstream from the dam, i.e. for X<0

Depth = 0.02m downstream from the dam, i.e. for X>0

In Test 6B: Depth = 8m upstream from the dam, i.e. for X<0

Depth = 0.4m downstream from the dam, i.e. for X>0

4. Parameter values

No preferred value of the eddy viscosity is specified.

In Test 6A:

Manning's n: 0.01 (uniform), as specified in Soares-Frazao and Zech, 2002.

Model grid resolution: 0.1m

(or ~36000 nodes in area bounded by vertical walls)

Time of end: the model is to be run until time t = 2 min (if an alternative end time is used run times must be reported for t=2 min)

In Test 6B:

Manning's n: 0.05 (uniform).

Model grid resolution: 2m

(or ~36000 nodes in area bounded by vertical walls)

Time of end: the model is to be run until time t = 30 min (if an alternative end time is used run times must be reported for t=30 min)

5. Required output

Software package used: version and numerical scheme.

Specification of hardware used to undertake the simulation: processor type and speed, RAM.

Minimum recommended hardware specification for a simulation of this type.

Time increment used, grid resolution (or number of nodes in area modelled) and total simulation time to specified time of end.

Value of eddy viscosity coefficient used.

From Test 6A:

Plots of the **water level** elevation versus time and **velocity** (scalar) versus time at locations G1 to G6 in Figure (a). Output frequency 0.1s. Coordinates provided as part of dataset.

Raster grids (or TIN) at the model resolution consisting of:

a. **Peak** water **elevations** reached during the simulation
b. **Peak velocities** (scalar) reached during the simulation
c. **Water elevation at times** 1, 2, 3, 4, 5, 10, 15, 20, 25 and 30 seconds.

From Test 6B:

Plots of the **water level** elevation versus time and **velocity** (scalar) versus time at locations G1 to G6 in Figure (a). Output frequency 1s. Coordinates provided as part of dataset.

Raster grids (or TIN) at the model resolution consisting of:

d. **Peak** water **elevations** reached during the simulation
e. **Peak velocities** (scalar) reached during the simulation
f. **Water elevation at times** 1, 2, 3, 4, 5, 10, 15 and 20 minutes.

6. Dataset content

Description	File Name
Georeferenced Raster ASCII DEM at resolution 0.05m for Test 6A	Test6ADEM.asc
Georeferenced Raster ASCII DEM at resolution 1m for Test 6B	Test6BDEM.asc
Location of output points for Test 6A	Test6Aoutput.csv
Location of output points for Test 6B	Test6Boutput.csv

7. Additional comments

Participants are asked to provide model results **at least** for the grid resolution specified above.

Model results for one alternative resolution or mesh may also be provided.

Participants are asked to justify their reasons for not carrying out the test, or for carrying out the test using an alternative resolution.

8. References

IMPACT, 2005 Investigation of Extreme Flood Processes and Uncertainty. Final Technical Report.

SOARES-FRAZAO, S. AND ZECH, Y., 2002 Dam-break flow experiment: The isolated building test case. Available online at: http://www.impact-project.net/wp3_technical.htm

Test 7 – River and floodplain linking

1. Modelling performance tested

The objective of the test is to assess a package's ability to simulate fluvial flooding in a relatively large river, with floodplain flooding taking place as the result of river bank overtopping. The following capabilities are also tested: 1) the ability to link a river model component and a 2D floodplain model component, with volume transfer occurring by embankment/bank overtopping and through culverts and other pathways; 2) the ability to build the river component using 1D cross-sections; 3) the ability to process floodplain topography features supplied as 3D breaklines to complement the DEM[36].

2. Description

The site to be modelled is approximately 7 km long by 0.75 to 1.75 km wide, see Map 1, and consists of a set of three distinct floodplains (Maps 2, 3, 4) in the vicinity of the English village of Upton-upon-Severn, although the river Severn that flows through the site is modelled for a total distance of ~20km. Boundary conditions are a hypothetical inflow hydrograph for the Severn (a single flood event with a rising and a falling limb, resulting in below bankfull initial and final levels in the river (table provided), and a downstream rating curve (table provided). This poses a relatively challenging test through the need for the model to adequately identify and simulate flooding along separate floodplain flow paths, and predict correct bank/embankment overtopping volumes. The volume exchange takes place over natural river banks and/or embankments along which flood depths are expected to be small.

The site has been subjected to flooding on a number of occasions but it is not the intention to replicate an observed flood for this exercise, hence the boundary conditions have been designed to provide a suitable benchmarking case.

River channel geometry

The channel geometry is provided in the form of a text file with cross-sections labelled M013 to M054 (a separate csv file containing cross-section locations and spacing is provided). A uniform channel roughness value is used. Any head losses due to the plan geometry of the river (meanders) are ignored. Along some sections the channel is adjacent to floodplains on just one or on both sides. 3D "breaklines" are provided which define a) the boundary between the river channel and the area expected to be modelled in 2D, and b) elevations along these boundaries (these are consistent with the DEM elevations). These elevations are to be used in the prediction of bank/embankment overtopping. Wherever no floodplain is modelled along the river channel (more than 50% of the total length of river banks), a "**glass wall**" approach (or equivalent) should be applied if water levels exceed the bank elevation in the cross-section (i.e. the water level rises above the bank without spilling out of the 1D model).

[36] The breaklines provided were derived from the 1m DEM and are a 'vector' representation of important crest lines in the topography (including embankments). The ability to recognise these important crest lines and apply the right elevations is tested, rather than the ability to process the 3D breaklines themselves.

A bridge at the North end of Upton (between cross-sections M033 and M034), for which no data are provided, is ignored. No other structure is known to affect the flow along the modelled reach of the river.

Floodplains

The extents of the three modelled floodplains are defined as follows (See Maps 2, 3, 4):

Floodplain 1: on West bank of the River, from upstream from Cross-Section M024, to upstream from M030 (floodplain breakline number 2, see below).

Floodplain 2: on East bank of the river, from upstream from Cross-Section M029, to upstream from M036.

Floodplain 3: on West bank of the river, from half-way between cross-sections M031 and M032 to half-way between cross-sections M043 and M044. This includes the "island" on which the village of Upton lies.

The floodplains are otherwise bounded by the river bank breaklines provided, see above in "River channel geometry". Away from the river, for consistency in model extent, it is suggested to draw the boundaries of the 2D models approximately along the 16m contour line.

Floodplain 3 has a physical opening below the 16m altitude along the Pool Brook stream to the North-West of Upton. The model should extent to the edge of the DEM in this location. (however this boundary is to be treated as closed, i.e. no flow)

Note that the narrow strip of floodplain (between FP 1 and FP 3) on the West bank of the river in the vicinity of cross-sections M030 and M031 does not need modelling in 2D. Cross-sections M030 and M031 have been extended as far as the hillside to the West.

A shapefile containing polylines defining the outer boundaries of the floodplains is provided.

A number of features in the floodplains are expected to impact on results significantly and will be modelled. This includes:

- embankments and elevated roads, for which **3D breaklines** are provided as part of the dataset. These can be used to adjust nodes elevations in the computational grid. They should be distinguished from the river/floodplain boundary breaklines mentioned in the previous section.

- a set of low bridges of total width ~40m under the elevated causeway (**A4104 road**) immediately west of Upton. This can be modelled as a single 40m opening through the A4104 causeway (elevations provided as floodplain breakline number 7). A photograph and a datafile containing various parameters (including X Y coordinates and dimensions) are provided as part of the dataset.

The modelled flood is not expected to inundate roads and built-up areas to any significant extent. Therefore a uniform roughness value is applied across the floodplains, with a specified value. The floodplain land use in this reach is predominately pasture with a lesser amount of arable crops. Any effect of buildings are ignored (for example in the town of Upton).

Any feature of the floodplain not mentioned above, including any perceived “false blockages” should be ignored. 2 “marinas” within floodplain 1 (near North end) and floodplain 2 (near South end) should simply be modelled as ground, with elevations as given by the DEM.

1D-2D volume transfer

No parameter value or modelling approach is specified for the prediction of river/floodplain volume transfer (except the elevations specified by the breaklines).

At the real site volume exchange between the channel and the floodplains also occur through a number of flapped outfalls. These are ignored.

A masonry **culvert** immediately upstream from the village of Upton (“Pool Brook”) is however modelled, see Map 4. It is assumed circular in cross-section. A photograph and a spreadsheet containing various parameters (including X Y coordinates and dimensions) are provided as part of the dataset.

An opening in the embankment (floodplain breakline number 2) at location X=384606 Y=242489 (see Map 2) at the southern end of Floodplain 1 (blocked by a **sluice** in reality) is assumed to remain opened during the duration of the flood. This should be understood as a 10m wide opening (invert level 10m) offering a pathway from Floodplain 1 to the river at cross-section M030.

Misc

The DEM is a 1.0m resolution LiDAR Digital Terrain Model (no vegetation or buildings) provided by the Environment Agency (http://www.geomatics-group.co.uk). Due to the very large size of the 1m DEM file, a coarsened 10m DEM is also provided, but it is emphasised that this is unlikely to provide the right elevations along embankments, river banks and other features, for which 3D breaklines are provided.

Minor processing of the original EA LiDAR DEM was done, consisting in merging tiles and filling small areas of missing data in the modelled floodplains. Areas of missing data (-9999) may remain in the DEM, but only outside the modelled 2D domain described previously.

The model is run until time T = 72 hours to allow the flood to settle in the lower parts of the modelled area.

3. Boundary and initial conditions

River channel:

Upstream: inflow versus time applied at the northernmost cross-section, cross-section M013.

Downstream: rating curve (flow versus head), applied at the southernmost cross-section, cross-section M054.

Initial condition: a uniform water level of 9.8m.

Floodplains:

Linked to the river channel along the river bank breaklines provided, and through the Pool Brook culvert (Floodplain 3) and the opening (sluice) at the South end of Floodplain 1.

All other boundaries are closed (no flow).

Initial condition: A uniform water level of 9.8m.

Pool Brook culvert: Initial water level 9.8m.

4. Misc. parameter values

Manning's n: 0.028 uniformly in river

0.04 uniformly in floodplains

Model grid resolution: 20m

(or ~16700 nodes in the model extent defined in Section 2 under "Floodplains")

Time of end: the model is to be run until time t = 72 hours (if an alternative end time is used run times must be reported for t=72 hours)

5. Required output

Software package used: version and numerical scheme.

Specification of hardware used to undertake the simulation: processor type and speed, RAM.

Minimum recommended hardware specification for a simulation of this type.

Time increment used, grid resolution (or number of nodes in area modelled) and total simulation time to specified time of end.

Raster grids (or TIN) at the model resolution consisting of:

a. **Peak water level elevations**
b. **Peak water depths**
c. **Peak velocities**
d. **Water level elevations at T=72hours.**
e. **Water depths at T=72hours.**

The above concerns the *floodplains* only

Water level elevation and **Velocity** versus time (output frequency 60s), at locations shown in Maps 2, 3, 4. Coordinates provided as part of the dataset.

Water level elevation and **Velocity** versus time (output frequency 60s) at the following river cross-sections (1D model):

M015

M025

M035

M045

6. Dataset content

Description	File Name
Georeferenced Raster ASCII DEM at resolution 1m	Test7DEM.asc
Georeferenced Raster ASCII DEM at resolution 10m	Test7DEM_10m.asc
1D Model Cross-sections	Test7-1DXS.txt
1D Model Cross-section locations and spacing	Test7-1DLoc-Spacing.csv
Location of output points	Test7-Output.csv
River bank breaklines	Test7-bank-bklines.csv
Floodplain breaklines	Test7-FP-bklines.csv
Photograph showing Pool Brook culvert	Test7-PoolBrookCulvert.jpg
Pool Brook culvert parameters	Test7-PoolBrookCulvert.xls
Photograph showing A4104 bridge	Test7-A4104bridge.jpg
A4104 bridge parameters	Test7-A4104bridge.xls
Dowstream rating curve (flow versus water level)	Test7-DSRatingCurve.csv
Upstream inflow (flow versus time)	Test7-USInflow.csv

Notes:

1D Model Cross-sections file (Test7-1DXS.txt): this contains 1 table of 6 columns for each cross-section. The first (chainage in m) and second (elevation in m) columns only should be used. All other data can be disregarded. The location and spacing of cross-sections are contained in file Test7-1DLoc-Spacing.csv

All coordinates in the British coordinates system.

7. Additional comments

Modelling instructions for this test have been provided as clearly as possible. Participants may contact Heriot-Watt University for more specific instructions. However it is intended that any aspect of the modelling not considered in this specification is left to the modeller's own initiative.

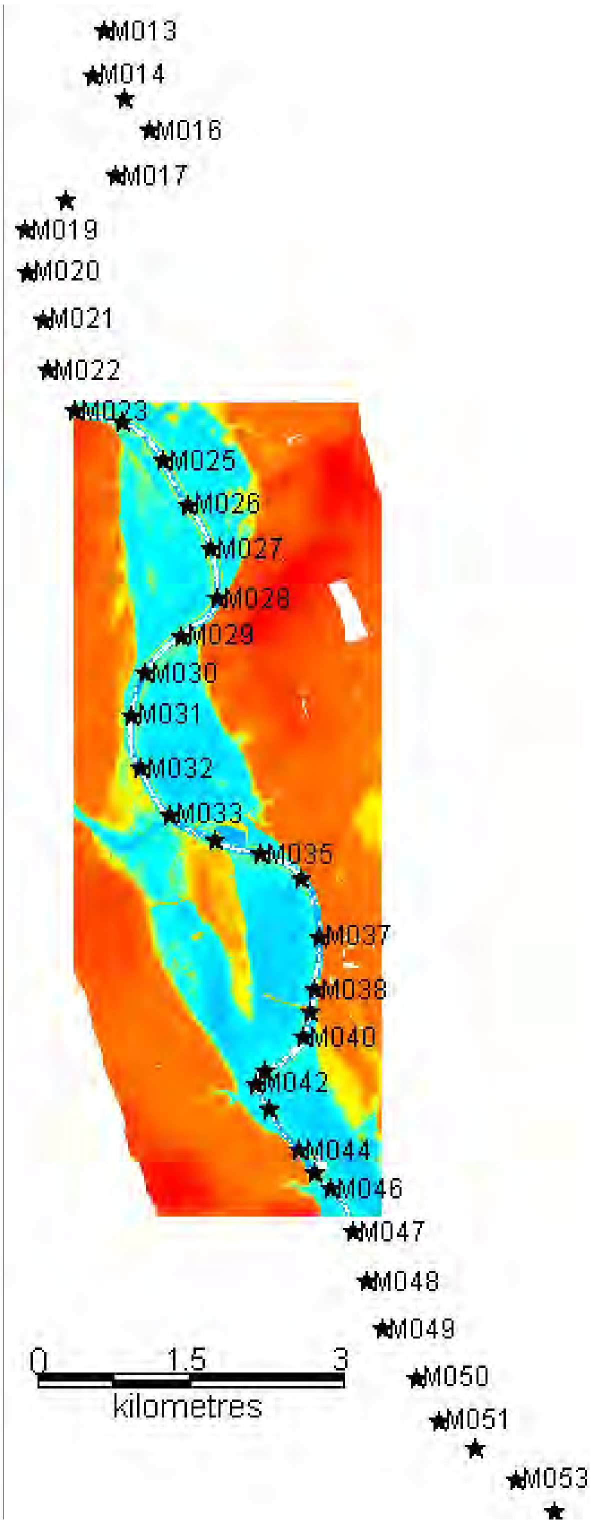
Linear interpolation should be used to interpolate inflow values.

Participants are asked to provide model results **at least** for the grid resolution specified above.

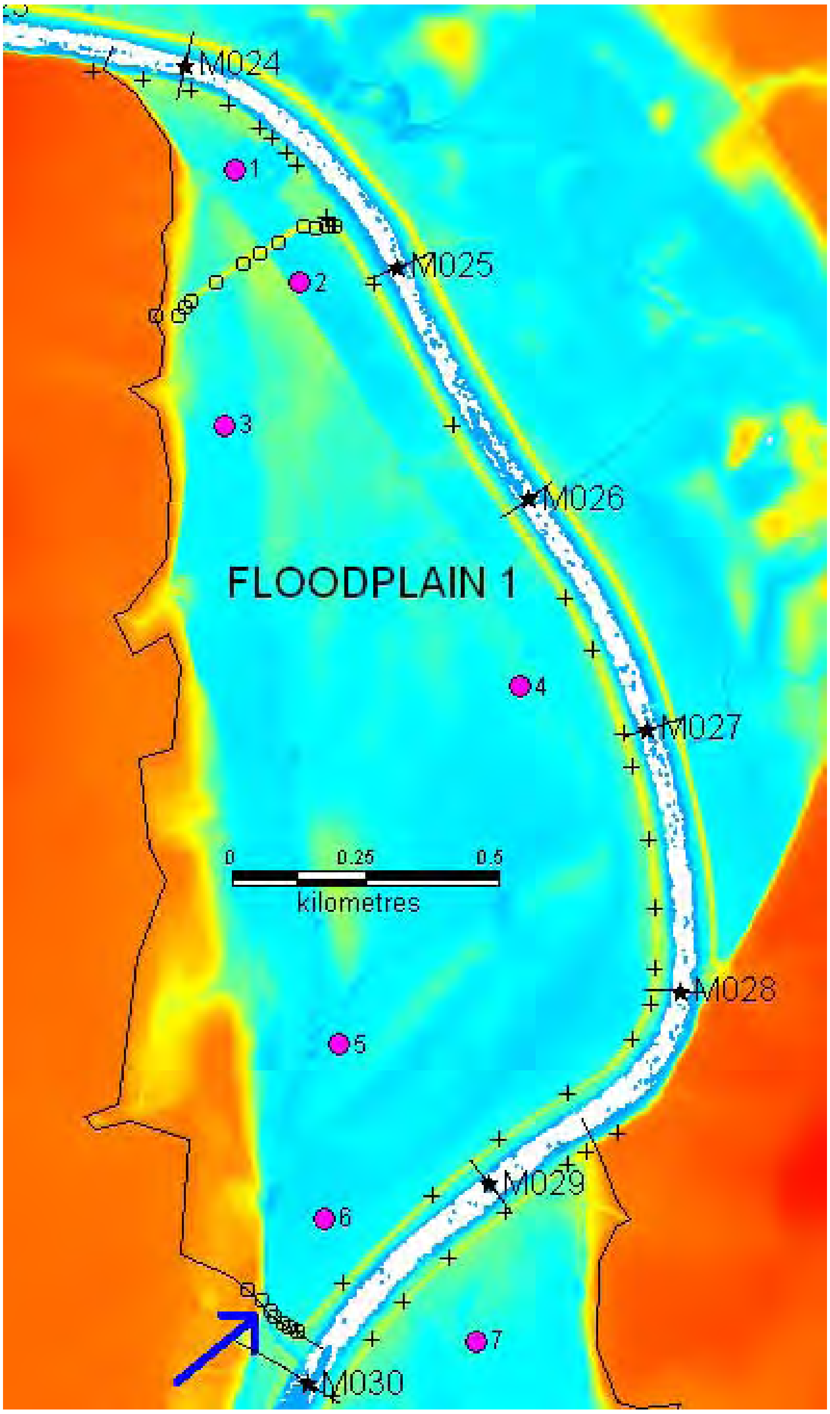
Model results for one alternative resolution or mesh may also be provided.

Participants are asked to justify their reasons for not carrying out the test, or for carrying out the test using an alternative resolution.

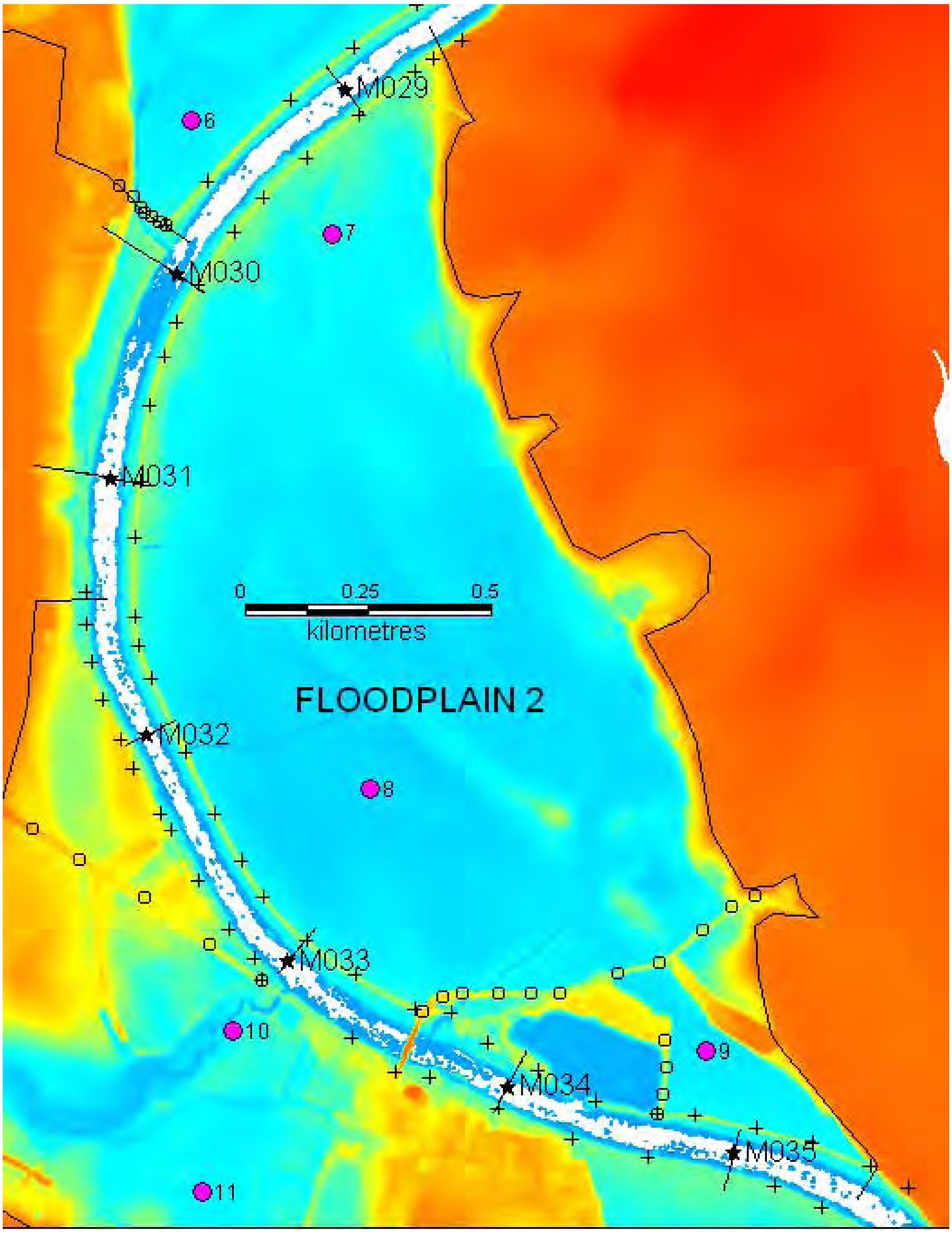
8. Maps



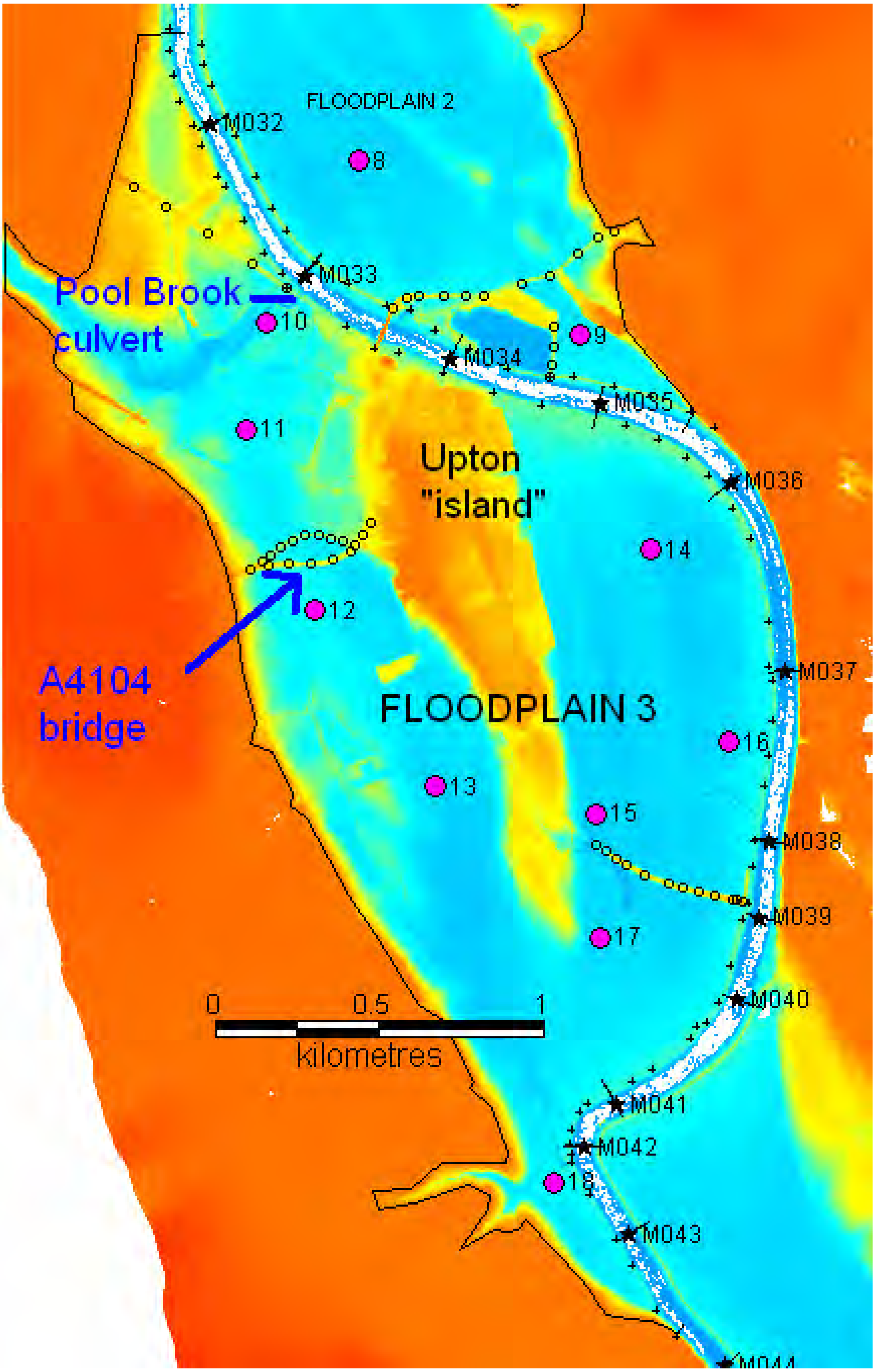
Map 1: Map of the modelled reach of the River Severn and floodplain system around Upton-upon-Severn. *The river flows from North to South.*



Map 2: Map of floodplain 1. Blue arrow: opening in embankment (sluice). Crosses: bank breaklines vertices. Circles: floodplain breakline vertices. Purple dots: output points. Black line: outer extent of model.



Map 3: Map of floodplain 2. Crosses: bank breaklines vertices. Circles: floodplain breakline vertices. Purple dots: output points. Black line: outer extent of model.



Map 4: Map of floodplain 3. Crosses: bank breaklines vertices. Circles: floodplain breakline vertices. Purple dots: output points. Black line: outer extent of model.

Test 8A – Rainfall and point source surface flow in urban areas

1. Modelling performance tested

This tests the package's capability to simulate shallow inundation originating from a point source and from rainfall applied directly to the model grid, at relatively high resolution.

2. Description

The modelled area is approximately 0.4 km by 0.96 km and covers entirely the DEM provided and shown in Figure (a). Ground elevations span a range of ~21m to ~37m.

The flood is assumed to arise from two sources:

- a uniformly distributed rainfall event illustrated by the hyetograph in Figure (b). This is applied to the modelled area only (the rest of the catchment is ignored).

- a point source at the location represented in Figure (a), and illustrated by the inflow time series in Figure (c). (This may for example be assumed to arise from a surcharging culvert.)

The DEM is a 0.5m resolution Digital Terrain Model (no vegetation or buildings) created from LiDAR data collected on 13th August 2009 and provided by the Environment Agency (http://www.geomatics-group.co.uk).

Participants are expected to ignore any buildings at the real location (Cockenzie Street and surrounding streets in Glasgow, UK) and to carry out the modelling using the "bare-earth" DEM provided.

A land-cover dependent roughness value is applied, with 2 categories: 1) Roads and pavements; 2) Any other land cover type.

The model is run until time T = 5 hours to allow the flood to settle in the lower parts of the modelled domain.

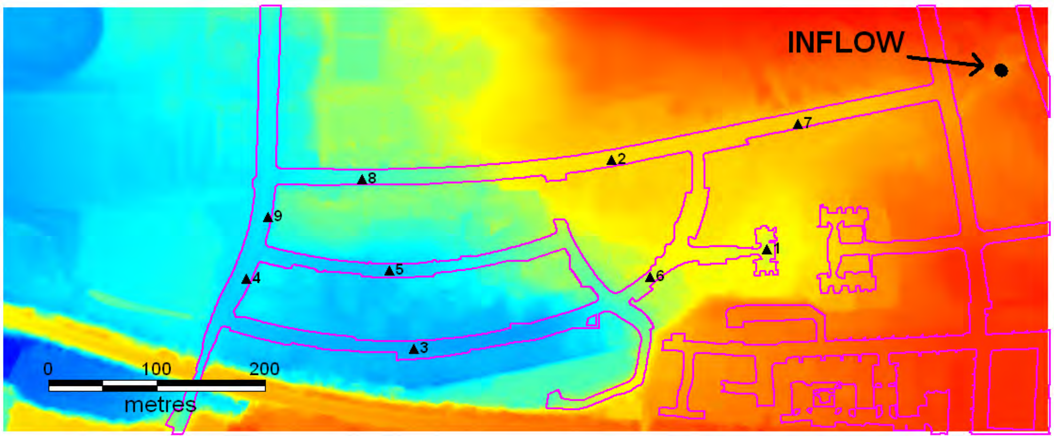


Figure (a): DEM used, with the location of the point source. Purple lines: outline of roads and pavements. Triangles: output point locations.

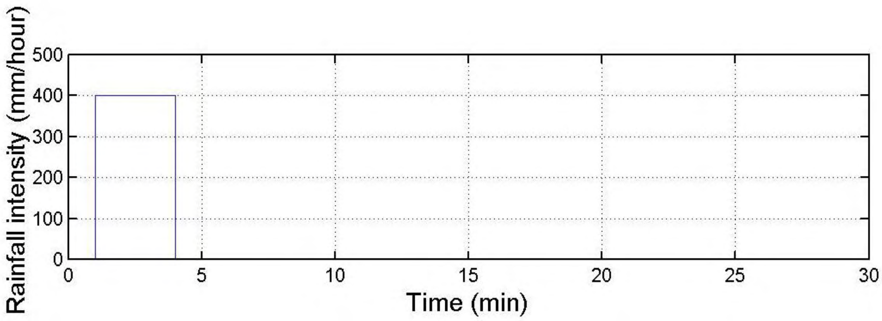


Figure (b): Hyetograph applied in Test 8A.

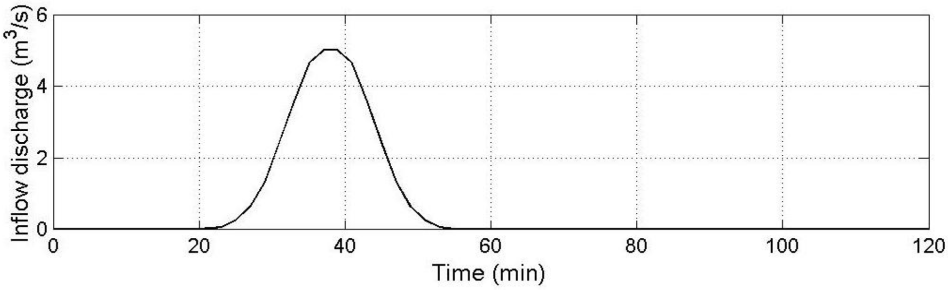


Figure (c): Inflow hydrograph applied in Test 8A at point location shown in Figure (a).

3. Boundary and initial conditions

Rainfall as described above. Hyetograph provided as table in dataset.

The point source is applied as described above. Coordinates and time series provided as part of dataset.

All boundaries of the modelled area are closed (no flow).

Initial condition: Dry bed.

4. Misc. parameter values

Manning's n: **0.02** for roads and pavements

0.05 everywhere else

Model grid resolution: **2m**

(or ~97000 nodes in the 0.388 km^2 area modelled)

Time of end: the model is to be run until time t = 5 hours (if an alternative end time is used run times must be reported for t=5 hours)

5. Required output

Software package used: version and numerical scheme.

Specification of hardware used to undertake the simulation: processor type and speed, RAM.

Minimum recommended hardware specification for a simulation of this type.

Time increment used, grid resolution (or number of nodes in area modelled) and total simulation time to specified time of end.

Raster grids (or TIN) at the model resolution consisting of:

a. Peak **water level elevations** reached during the simulation
b. Peak water **depths** reached during the simulation

Water level elevation and **Velocity** versus time (output frequency 30s), at locations shown in Figure (a) and provided as part of the dataset.

6. Dataset content

Description	File Name
Georeferenced Raster ASCII DEM at resolution 0.5m	Test8DEM.asc
Rainfall hyetograph (rainfall intensity vs. time)	Test8A-rainfall.csv
Point source boundary condition table (inflow vs. time)	Test8A-point-inflow.csv
Point source coordinates	Test8A-inflow-location.csv
Location of output points	Test8Output.csv
Outline of roads and pavements (shapefile polygons)	Test8Road_Pavement_polyg_region
Outline of roads and pavements (ASCII raster file)	Test8RoadPavement.asc

7. Additional comments

The location modelled is in the City of Glasgow, UK (Cockenzie Street and surrounding streets)

Linear interpolation should be used to interpolate inflow values and rainfall intensity values.

Participants are asked to provide model results **at least** for the grid resolution specified above.

Model results for one alternative resolution or mesh may also be provided.

Participants are asked to justify their reasons for not carrying out the test, or for carrying out the test using an alternative resolution.

Test 8B – Surface flow from a surcharging sewer in urban areas

1. Modelling performance tested

This tests the package's capability to simulate shallow inundation originating from a surcharging underground pipe, at relatively high resolution. The pipe is modelled in 1D and connected to the 2D grid through a manhole.

2. Description

The modelled area is approximately 0.4 km by 0.96 km and covers entirely the DEM provided and shown in Figure (a). Ground elevations span a range of ~21m to ~37m.

A culverted watercourse of circular section, 1400mm in diameter, ~1070m in length, and with invert level uniformly 2m below ground is assumed to run through the modelled area. An inflow boundary condition is applied at the upstream end of the pipe, illustrated in Figure (b). A surcharge is expected to occur at a vertical manhole of $1m^2$ cross-section located 467m from the top end of the culvert, and at the location shown in Figure (a).

The flow from the above surcharge spreads across the surface of the DEM.

The DEM is a 0.5m resolution Digital Terrain Model (no vegetation or buildings) created from LiDAR data collected on 13th August 2009 and provided by the Environment Agency (http://www.geomatics-group.co.uk).

Participants are expected to take into account the presence of a large number of buildings in the modelled area. Building outlines are provided with the dataset. Roof elevations are not provided (arbitrary elevations to be set by modellers if needed, at least 1m above ground).

A land-cover dependent roughness value is applied, with 2 categories: 1) Roads and pavements; 2) Any other land cover type.

The model is run until time T = **5** hours to allow the flood to settle in the lower parts of the modelled area (or until this has happened according to the model)

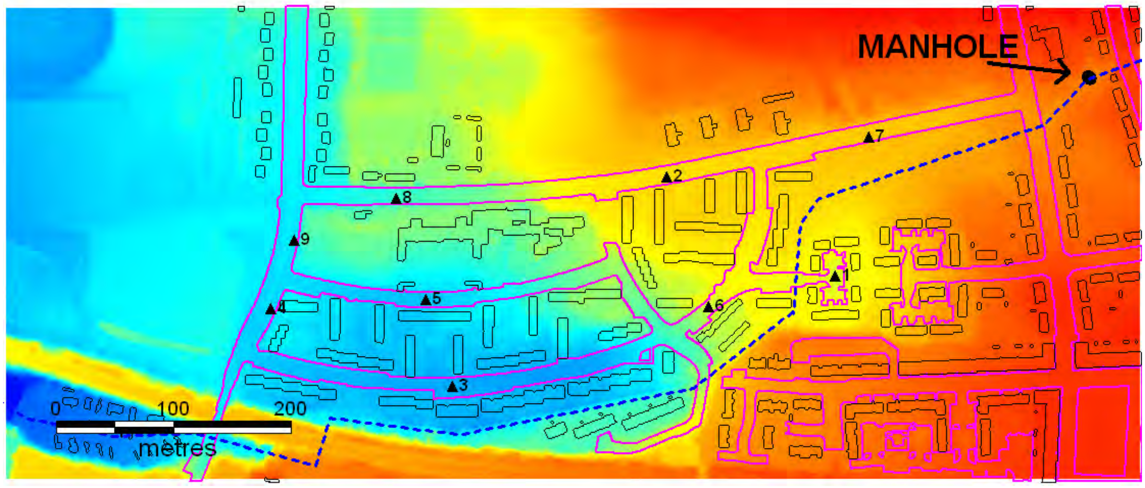


Figure (a): DEM used, with the location of the manhole. The course of the underground pipe is indicated, although irrelevant to the modelling. Purple lines: outline of roads and pavements. Black lines: building outlines. Triangles: output point locations.

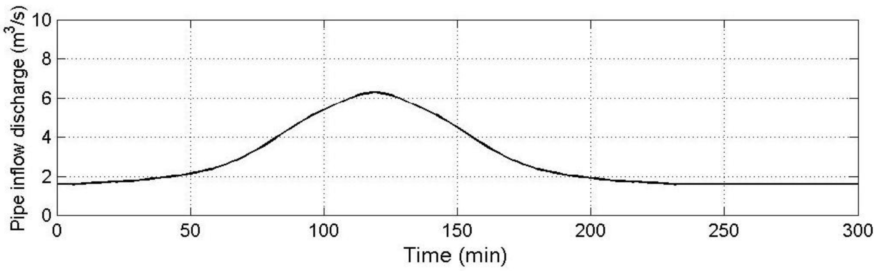


Figure (b): Inflow hydrograph applied in Test 8B at upstream end of culvert.

3. Boundary and initial conditions

Underground pipe

- Upstream boundary condition: discharge versus time provided as part of dataset

- Downstream boundary condition: free outfall (critical flow)

- Baseflow (uniform initial condition): 1.6 m^3/s

2D domain

Manhole connected to 2D grid in one point.

All boundaries of the modelled area are closed (no flow).

Initial condition: Dry bed.

Conditions at manhole/2D surface link

The surface flow is assumed not to affect the manhole outflow.

4. Misc. parameter values

Manning’s n: **0.02** for roads and pavements

0.05 everywhere else

Model grid resolution: **2m**

(or ~97000 nodes in the 0.388 km^2 area modelled)

Time of end: the model is to be run until time t = 5 hours (if an alternative end time is used run times must be reported for t=5 hours)

5. Required output

Software package used: version and numerical scheme.

Specification of hardware used to undertake the simulation: processor type and speed, RAM.

Minimum recommended hardware specification for a simulation of this type.

Time increment used, grid resolution (or number of nodes in area modelled) and total simulation time to specified time of end.

Raster grids (or TIN) at the model resolution consisting of:

a. Peak **water level elevations** reached during the simulation
b. Peak water **depths** reached during the simulation

Water level elevation and **Velocity** versus time (output frequency 30s), at locations shown in Figure (a) and provided as part of the dataset.

Discharge versus time through the manhole (output frequency 30s).

6. Dataset content

Description	File Name
Georeferenced Raster ASCII DEM at resolution 0.5m	Test8DEM.asc
Culvert upstream boundary condition table (discharge vs. time)	Test8B-pipe-inflow.csv
Geometry of pipe	Test8BPipeGeometry.xls
Location of output points	Test8Output.csv
Outline of roads and pavements (shapefile polygons)	Test8Road_Pavement_polyg_region
Outline of roads and pavements (ASCII raster file)	Test8RoadPavement.asc
Outline of buildings (shapefile polygons)	Test8Buildings_polyg_region
Outline of buildings (ASCII raster file)	Test8Buildings.asc

7. Additional comments

The location modelled is in the City of Glasgow, UK (Cockenzie Street and surrounding streets). The above representation of the culverted watercourse is a gross simplification of reality and is for the purpose of the present test only.

Linear interpolation should be used to interpolate inflow values.

Participants are asked to provide model results **at least** for the grid resolution specified above.

Model results for 1 alternative resolution or mesh may also be provided.

Participants are asked to justify their reasons for not carrying out the test, or for carrying out the test using an alternative resolution.

APPENDIX B – Misc. comments provided by participants

The following are comments that the participating software developers provided as part of their submission of results, with the intention that they should be considered when interpreting the test results. They concern most or all tests.

1. FloodFlow

FloodFlow is commercially available as part of the WinDes suite of programs.

"The following is an extract from our software help:

'Although the general term for this type of analysis is "shallow water" it must be remembered that it was first developed for offshore coastal flows. The analysis needed for the urban and rural, pluvial and fluvial flooding would be better described as "sheet flow" and the very significant "ground effect" must not be ignored, as it can be the most significant variable.

It is well known that Manning's n is dependent on depth. In ordinary concrete pipes n can vary by 20% from half full to full flow. This effect is often ignored in open channel analysis particularly if the depth of flow does not vary greatly for the selected case. However in sheet flow analysis n may vary by an order of magnitude between 100mm and 500mm depth. This variation therefore is of primary engineering significance for sheet flow and FloodFlow takes this into account.

The recommended Mannings n for a grass swale (Ciria Suds manuals) flowing at 100mm deep is 0.3 while a regular shaped, low vegetation channel would require a Mannings n of 0.03 (Ven Te Chow 1959). This implies that when flow becomes very shallow (i.e. sheet flow) Mannings n needs to vary from 0.3 increasing to 0.03 as the flow deepens.

A similar assumption can be made for paved areas. The analysis is based on a grid. The base of each grid is flat and represents the average level of that grid square. This assumption has an acceptable error associated with it when the flow is say 1.0m deep but it is unreasonable to assume that flows 100mm deep fill the whole square and are not significantly obstructed by kerbs, paths, road cambers, obstructions etc.

These ground effects have been taken into account by gradually varying Manning's n with depth. The following table represents the n values chosen for a paved surface.

Depth (m)	n
0.1	0.1
0.25	0.03
0.5	0.012
1	0.01

A similar relationship for grass can be determined by factoring the above table by 3 while the mixed surface option factors it by 2.'"

Note: Micro Drainage provided results from simulations using the above depth-dependent roughness approach (all using the 'Urban' option described above). Results were therefore not directly comparable to others, and are presented separately in Appendix D. Additional results were provided for Test 1 based on a constant roughness approach as specified.

2. Flowroute

"The development and validation of Ambiental's Flowroute™ flood modelling software platform was initiated in 2004 in collaboration with flood scientists at Cambridge University. The platform was designed specifically to simulate river dynamics and floodplain inundation in complex, congested urban areas in a highly computationally efficient manner. Recent versions of the code have been optimised for wide-area, (e.g. national-scale) applications, enabling processing of domains of up to 240 million cells, whilst including multiple specifications of flood source locations and types.

The two-dimensional (2-D) component of Flowroute™ employs a diffusion wave, explicit, finite difference solver to simulate river, coastal and surface water flood flows. The model can also receive or lose water from point source inflows (e.g. defence breach / overtopping, surcharging sewers, pipe burst) and outflows (e.g. sewers, watershed pour points, pumps).

The surface water flood modelling module of Flowroute™ (see Butler *et al.,* 2008) uses numerical discretisation in space and time. The floodplain is treated as a grid of square cells, with flow occurring between edge-connected cells at each time step. The continuity and Manning's equations are solved to calculate the flow rate. The continuity equation relates flow across cell boundaries to the volume stored in the cell, and Manning's equation relates flux to surface slope and hydraulic radius.

The OpenMP (Open Multi-Processing) version of the code, Flowroute-OMP, which is deployed here, can be run within a distributed or 'cloud' computing environment, and is capable of simulating flood flows across multi-million cell domains in parallel. With regards to several of the tests carried out as part of this benchmarking study, it should be noted that some of the efficiency gains achieved when using the software for 'real world' simulations over wide areas are not realised when processing small, synthetic test areas due to the specification of certain parallel processing routines. Further, for the purposes of model testing, relatively small model time steps were used so as to maximise stability throughout. Where required, order of magnitude reductions in run-time are achievable for wide area, high resolution applications using alternative model parameterisations and IT configurations. A variety of proprietary pre- and post-processing routines can also be used to maximise the efficiency of the flood modelling workflow as part of complex flood modelling applications.

Previous versions of Flowroute™ also incorporate a one-dimensional (1-D) channel model which uses the kinematic approximation to the Saint-Venant equations to describe 1-D unsteady open channel flow. These equations consist of a continuity equation and a momentum equation, and can be used to predict the location and magnitude of overbank breach points. Future releases of Flowroute™ will allow for coupling of output from the 1-D channel model with the latest version of the 2-D floodplain inundation model so as to provide the facility to model dynamic interactions between channel and floodplain flows in an efficient manner."

“Output of velocity grids is not included within the version of Flowroute used for this exercise. This feature will be included within future releases. “

3. InfoWorks 2D

“InfoWorks 2D (IW2D) is a fully hydrodynamic finite volume model, which solves the shallow water equations. It is based on the Godunov numerical scheme and the so-called Riemann solvers. The solver is fully conservative and shock capturing, so it can deal with any changes in flow regime. The model is, therefore, particularly suitable for the simulation of rapidly varied flows, such as those occurring in typical flooding events.
IW2D uses unstructured meshes, which makes the model fully flexible from the geometric point of view.”

4. ISIS 2D

“The software package used in this report is ISIS 2D, a commercially available software application with the following solvers:

- The ADI Solver (Alternating Direction Implicit) with a focus on simulating subcritical flows
- The TVD Solver (Total Variation Diminishing) with a focus on simulating supercritical flows and therefore offering a shock-capturing capability
- The FAST Solver for a rapid solution of flows dominated by topography, where depression filling is the main mechanism.

A free version of ISIS 2D with limited domain size and ADI solver only is available to download from www.isisuser.com. ”

“ In general, the Alternating Direction Implicit (ADI) solver is suitable for simulating subcritical flows and Total Variation Diminishing (TVD) for problems needing shock-capturing due to supercritical flow regimes. However, the ADI solver can still be used for supercritical flows when their occurrence is localised and brief. Use of the ADI solver in these circumstances will be at the expense of some accuracy, although careful use of the eddy-viscosity parameters may allow unphysical fluctuations in the model solution to be reduced. The FAST solver may provide rapid solutions for flows dominated by topography where depression filling is the main mechanism.”

5. JFLOW-GPU

"JFLOW-GPU has been used to provide solutions for the 2D hydraulic model benchmarking exercise. JFLOW-GPU solves the 2D diffusion wave equation which is obtained by simplifying the 2D shallow water equations. The flow is driven by the balance between surface slope and bed friction. JFLOW-GPU solves the resulting equations on a Graphics Processing Unit (GPU). The GPU offers high performance parallel processors which enable computations to be performed more quickly than would otherwise be possible on the CPU."

"It should be noted that the JFLOW model is designed to run problems using real world data and certain features of the model reflect this. Depth and velocity data can only be extracted at 1 minute intervals (or multiplies thereof) and so it has not been possible to provide monitor data at the suggested output intervals for Test 3, 4 and 6B. The peak data are constructed from these intermediate grids and thus inevitably reflect the sampling interval used. If this interval is inappropriate for the flow problem at hand, then the 'maximum' results are unlikely to provide a good representation of reality. The monitor point data is provided in separate spreadsheets for each test case.

The velocities obtained from JFLOW-GPU are considered indicative and should be viewed as such."

6. MIKE FLOOD

"All tests have been simulated with the suggested model parameters and no additional
simulations with alternative model parameters have been made.

If the tests have been designed solely to evaluate the computational accuracy of the numerical hydrodynamic engines, the necessary rationalisation of the raw digital height data to a reduced grid resolution could potentially compromise the integrity of the input data and introduce an additional level of uncertainty in the test results. There is a concern that any steps undertaken, by all software suppliers, to transform the DEM data will cause a degree of variation in test results (e.g. water levels, velocities, etc) that would compromise detailed comparison of the simulations engines.

We have assessed various ways to transform from the grid size in which the DEM data was supplied to the coarser grid size requested for the simulations. We have chosen to a resample method which essentially calculates the elevation in a coarse cell as the average of the finer cells within the course cell.

The selection of appropriate flood and dry values is important. While this should not significantly affect maximum predicted flood water levels, slight variations in the modelled flood extents may be evident in the results of different software packages.

Where point series results have been requested, several tests locate output points at the interface of grid cells. MIKE 21 provides point results at the centre point of each 'wet' cell, and it is not always possible to select a single cell nearest to the desired output point. As such, some interpretation has been necessary which may lead to further variations in results between the evaluated software packages."

Our response: All the issues raised by DHI are valid. Differences between models arising due to the different ways in which node/element elevations are assigned (although the DEMs for Tests 1,2,3,4,5 were designed to make such effects negligible), or the different ways in *which output time series can be interpolated from results at nodes are also of interest. In fact they are also inevitable due to the many discretisation approaches used by the different models.*

7. RFSM

"The Rapid Flood Spreading Model (RFSM) was developed more than 10 years ago as a simple and fast flood routing hydraulic model to be used in the context of probabilistic Flood Risk Analysis where thousands of runs are necessary to account for a large range of return periods, parameter values and uncertainties. Its strengths are a fast computational time and a quick and easy model set-up, which also make it a suitable tool for national scale flood mapping.

Two versions of RFSM have been tested during this project:

- the Direct RFSM, which is the RFSM used in NaFRA and MDSF2 at the time of writing
- the Dynamic RFSM, which is a time-stepped version of the RFSM, recently developed at HR Wallingford

The Direct RFSM is a simplified hydraulic model that takes as input flood volumes discharged into floodplain areas from breached or overtopped defences and spreads the water over the floodplain while accounting for the topography.

This flood cell-type model uses depressions (Impact Zones) in the topography as computational elements. The excess volume is spilled from an Impact Zone (IZ) to another depending on Communication Levels until there is no more excess volume in any IZ. This process is not time-linked and provides only with the final water depths. Further details on the RFSM can be found in Mulet-Marti and Sayers (2006), Gouldby *et al* (2008) and Lhomme *et al* (2008). The model has been developed over the last five years to provide a fast solution to the flood spreading problem for use in probabilistic flood risk models.

The Dynamic RFSM is a new spreading model developed at HR Wallingford in 2009 in order to enhance the modelling capability of the RASP-RFSM system. It is a quasi-2D flood cells-type model calculating the evolution in time of the flood and takes as input discharge time-series. At each time-step, discharges between Impact Zones are calculating from the water levels using the Manning or the Weir relation (Cunge *et al* 1980), i.e. this is a diffusive wave-type model. Computational elements can be either depressions automatically calculated from the topography or polygons defined by the user. The time step is defined by the user.

Most of the tests featured in this 2D Benchmarking are designed for hydrodynamic models which aim to model the flow at a relatively fine scale (typical computational element dimension: 1–10 meter). The RFSM (both Direct and Dynamic) are designed to be run at a larger scale (typical computational element dimension: 50–300 meters).

Hydrodynamic models favour an accurate computation of the flow over fast computation (although they can be quick to run depending on the computer performance and the mesh characteristics) whereas simpler models like RFSM favour fast computation over accurate computation of the flow (although in some situations the diffusive wave formulation is a very good approximation of the shallow water equations).

For this reason in most tests the grid resolution used for the RFSM calculation is different from that advised in the Benchmarking documents.

The velocity computation is still under development. The numerical results for the velocity have not been given except on Test 5 where only velocity maps are given.

No calibration has been undertaken in the benchmarking process. The only parameters that were changed for the Dynamic RFSM were the Manning coefficient (as advised) and the time-step. For the Direct RFSM, default values of the friction and multiple spilling coefficients were used."

8. Flood Risk Mapper

Adapted from FRM 0.12 User's guide:

FRM simulates surface routing over a uniform 2D grid of square cells. The surface routing algorithm uses the following numerical scheme:

$$\Delta d_{ij} = \Delta t \, [\, (\sum Q_{ij,k}) - Q_{ij,infiltration} \,] \, / \, A$$

At each timestep, the net flow rate entering or leaving each cell is computed. The net flow rate for a cell is comprised of four inter-cell flow rates ($Q_{ij,k}$ where k = { North, South, East, West }) computed using Mannings formula applied in 2D, an infiltration flow rate $Q_{ij,infiltration}$ leaving through the base of the cell, and for the source cell only the source flow rate Q_{source}. The net flow rate entering or leaving each cell is then used to compute the change in depth (Δd_{ij}) for each cell.

Flow leaving a cell through infiltration is considered lost from the surface routed flow.

Volume of flow is conserved during a surface routing simulation and is limited only by numerical precision.

The surface routing algorithm uses an explicit numerical algorithm for time-stepping. The maximum allowable time-step is user-configurable. However, irrespective of the user-specified time-step, where necessary, the algorithm uses an adaptive sub time-step to attempt to accelerate the calculation whilst also preserving accuracy.

The default time-step of 30 seconds should be suitable for most applications of FRM.

Also:

An issue with how the velocity data is being written out of the programme was identified during the project. Velocity information was consequently not provided.

9. TUFLOW

TUFLOW uses a 2D ADI finite difference solution of the full Shallow Water Equations, including sub-grid scale turbulence (viscosity term). The 2D ADI scheme has been adapted to represent upstream controlled flow regimes for supercritical and weir flow between cells. TUFLOW's 1D solution is an explicit solution of the full St Venant equations. There are several mechanisms for linking the 1D and 2D schemes. The two most commonly used are: (a) the transfer of the 1D water level profile along a 1D open channel to the 2D floodplain (with a flow exchange across the 1D/2D interface back to the 1D scheme), and (b) a sink/source linkage between 1D manholes and other 1D structures with the overlying or adjacent 2D domain (with a water level exchange back to the 1D scheme). For more details refer to the publications at http://www.tuflow.com/Downloads_Publications.htm or contact support@tuflow.com.

TUFLOW 2D uses ground elevations at the cell centre and the cell mid-sides, so essentially each cell has five elevation points used in the hydraulic computations. The elevations at the cell corners are only used for output purposes. Whole cells or just the cell sides can wet and dry, with the default wet/dry depths being 2mm and 1mm respectively. For several of the test models the wet/dry depths were reduced to 0.2mm and 0.1mm due to the very shallow flows experienced during the simulation.

TUFLOW does not employ shock-capturing techniques. For the test models with "shock" waves (Tests 3, 6A and 6B), whilst TUFLOW remains stable and produces useful results it is not necessarily the most appropriate scheme if the focus is on the formation of hydraulic jumps and other complex flow formations. Shock capturing solutions such as that used by TUFLOW FV are generally better suited to problems of this type. However, for flood, dambreak and storm tide inundation modelling the detailed representation of hydraulic jumps, etc is usually not relevant, the main criteria being that software such as TUFLOW adequately represents these sections of upstream controlled flow to meet the modelling objectives.

Observations arising from the benchmarking that maybe of interest to TUFLOW modellers are:

1 2D SA inflow boundaries performed better than 2D QT boundaries, in terms of stability and mass error in the vicinity of the boundary, for those models with rapidly varying inflows onto a dry bed (Tests 2, 3, 4, 5). 2D SA boundaries typically allowed higher timesteps and faster run times with improved performance for these models. 2D QT boundaries are more suited to the inflow boundary of a river or stream where there is usually a base flow, and the flow varies across the inflow boundary according to variations in Manning's n and depth as the 2D QT boundary will automatically vary the flow distribution across the boundary according to the topography and bed roughness.

2 The number of iterations was set to 4 (rather than the default of 2) for models with rapidly varying flows (Tests 3, 5, 6A and 6B). The need to increase the number of iterations becomes apparent due to unacceptably high mass errors occurring.

10. TUFLOW FV

The TUFLOW-FV numerical scheme solves the conservative integral form of the Non-Linear Shallow Water Equations (NLWSE), including viscous flux terms and source terms for coriolis force, bottom-friction and various surface and volume stresses. The scheme is also capable of simulating the advection and dispersion of multiple scalar constituents within the model domain.

The spatial domain is discretised using contiguous, non-overlapping but irregular triangular and quadrilateral “cells”. Advantages of an irregular flexible mesh include:

- The ability to smoothly resolve bathymetric features of varying spatial scales;
- The ability to smoothly and flexibly resolve boundaries such as coastlines; and
- The ability to adjust model resolution to suit the requirements of particular parts of the model domain without resorting to a “nesting” approach.

The flexible mesh approach has significant benefits when applied to study areas involving complex coastlines and embayments, varying bathymetries and sharply varying flow and scalar concentration gradients.

A cell-centred spatial discretisation is currently employed in TUFLOW-FV, and requires the calculation of numerical fluxes across cell boundaries. As with many finite volume schemes non-viscous boundary fluxes are calculated using Roe’s approximate Riemann solver. Viscous flux terms are calculated using the traditional gradient-diffusion model with a variety of options available for the calculation of eddy-viscosity and scalar diffusivity. The Smagorinksy eddy-viscosity model and the non-isotropic Elder diffusivity model are the options most commonly adopted by BMT WBM modellers.

Both first-order and second-order spatial discretisation schemes are available in TUFLOW-FV. The first-order scheme assumes a piecewise constant value of each conservative constituent in a model cell. The second-order scheme assumes a 2D linear polynomial reconstruction of the conservative constituents within the cell (i.e. a MUSCL scheme). The Total Variation Diminishing (TVD) property (and hence stability) of the solution is ensured using a choice of gradient limiter schemes.

The second-order spatial reconstruction scheme allows for much sharper resolution of gradients in the conserved constituents for a given level of spatial resolution and was used in the runup modelling. This is important for resolving relatively short waves (e.g. tsunamis) without excessive numerical diffusion or without over-refining the spatial mesh discretisation. The numerical resolution of sharply varying current distributions and sharp scalar concentration fronts are also much improved with the second-order scheme.

Spatial integration is performed using a midpoint quadrature rule. Temporal integration is performed with an explicit Euler scheme and must therefore maintain a stable time step bounded by the Courant-Friedrich-Levy (CFL) criterion. A variable time step scheme is implemented to ensure that the CFL criterion is satisfied with the largest possible time step. Outputs providing information relating to performance of the model with respect to the CFL criterion are provided to enable informed refinement of the model mesh in accordance with the constraints of computational time.

In very shallow regions (~<0.05m depth), the momentum terms are dropped, in order to maintain stability as the NLSWE approach the zero-depth singularity. Mass conservation is maintained both locally and globally to the limit of numerical precision across the entire numerical domain, including wetting and drying fronts. A conservative mass re-distribution scheme is used to ensure that negative depths are avoided at numerically challenging wetting and drying fronts without recourse to adjusting the time step. Regions of the model domain that are effectively dry are readily dropped from the computations. Mixed sub/super-critical flow regimes are well handled by the FV scheme which intrinsically accounts for flow discontinuities such as hydraulic jumps or bores that may occur in trans-critical flows.

Transport of scalar constituents is solved in a fully-coupled fashion with the NLWSE solution. Simple linear decay and settling are optionally accommodated as source/sink terms in the scalar transport equations.

TUFLOW-FV accommodates a wide variety of boundary conditions, including those necessary for modelling the processes of importance to the present study:

- Water level time series;
- In/out flow time series;
- Bed friction;
- Coriolis force;
- Mean Sea Level Pressure gradients;
- Wind stress; and
- Wave radiation stress.

11. UIM

UIM is a 2D non-inertia overland flow model, which neglects the acceleration terms in the shallow water equations. The model adopts finite difference explicit scheme as numerical solver.

Adaptive time stepping function is included in UIM to avoid chequerboard oscillations, which is common in explicit models when a too big time step is applied (Hunter et al, 2006). However, the tolerance setting influences the model efficiency significantly. The highly restrictive setting may end up with un-realistic small time step but the improvement of accuracy is limited. The attributes of UIM limit its application to problems like Case 6 with sudden change of water surface. Although the model can run the simulation, the results are not realistic.

UIM model is also fully integrated with the 1D sewer network model SIPSON. The two models are linked by the discharge through manholes, therefore, the feature of 1D sewer and 2D overland linkage is available. Nevertheless, the 1D river channel and 2D overland flow linkage is still under development, due to the type of linkages is lines along the channel, rather than points. Hence, Case 7 was simulated by separate 1D ISIS and 2D UIM models, instead of an integrated software package. The model can be executed on both Window-based and Linux-based machines, and OPENMP is recently introduced into UIM for multi-processing.

12. References

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APPENDIX C – Modelling approaches used in Test 7

There was room for the modellers' own initiative in Test 7 on how to model a number of features. This section contains all information provided by the participants on modelling approaches. (note: any figure referred to is not reproduced)

1. FloodFlow

See Appendix D for results.

Use of depth-dependent roughness, see Appendix B.1.

"The terrain model was built from the 10m DEM."

"FloodFlow splits the 1D and 2D models horizontally, not vertically at the bank positions. Results from the 1D model only show water levels up to the top of the defined cross-section. The river cross-section results were compiled by manually combining the 1D and 2D results. A future version of the software will complete this operation automatically"

2. Infoworks 2D

"The mesh element size was reduced locally in the vicinity of the breaklines to ensure that the elevations were correctly represented in the mesh elements."

3. ISIS2D

"The Masonry Culvert Upstream of Upton (Pool Brook): The culvert at its floodplain end is expected to be connected to a watercourse but the DTM indicated a rather higher ground level at approximately 12.0mAOD, which would be above the soffit level of the culvert. After a number of sensitivity tests, we decided to remove the anomalous terrain feature from the floodplain and introduce a depression reflecting the watercourse, as visible from Google Earth. This enabled us to model the culvert as part of the 1D model, where the culvert is connected to a free flow weir on its floodplain end. The link between the 1D and 2D models was capable of representing flows in both directions.

The Elevated Causeway A4104 West of Upton Hydraulic Complex: The problem here is that 2D models do not intrinsically cope with the culvert, such as the flood relief culvert under the bridge in this road. An area, as shown in Figure 8.1, in this vicinity was therefore removed from the active 2D modelling area, which corresponded conveniently to an enclosed area within high grounds intersecting the embankment. A sub-model of this area was then built using ISIS. Therefore there is no double counting of volume or flow in this area and the results show that both 1D and 2D models smoothly interact with each other at both sides of the road. The oval enclosure was modelled as a reservoir unit in ISIS and conduit units with inverted Preissmann slots were used to represent very small flows during dry floodplain conditions.

Results may be different from other models for FP1 due to the fact we did not model the optional sluice at 10mAOD.

Defence crest elevations and links to 1D model have been reviewed to represent better the exchange of water between channel and floodplain for both wetting and drying phases.

To allow linking to controlling defence crest points, an interpolated 1D river unit at M024i01 has been added to represent the lowest point of bank line 1. Other points seem to be properly captured.

Further small discrepancies may be caused by different interpretations of the given data, e.g. where floodplain topographic breaklines overwrite defence crest elevations.

For floodplain 1, water levels drop to ~12.6m, as expected from the lowest point on the defence line for this floodplain (the optional opening in the embankment has not been modelled).

For floodplain 2, levels for points 7 and 8 drop to 12.5m as expected from defence crest elevations. Point 9 appears to be consistent with the other models.

For floodplain 3, points 10 and 11 show the expected behaviour, with levels following those in the channel via the Pool Brook culvert. The remaining points drop back to 11.52m fairly quickly, and then drop more slowly as water returns via a narrow flow path at the southern end of the floodplain. "

4. MIKE FLOOD

"1. DEM
The 1m DEM originally supplied has been used for generating a 20 m DEM.
2. River channel geometry
Cross sections have been applied as supplied. However, in order to smoothen the volume transfer between river and flood plain additional cross sections have been interpolated at approximately 100 meter distance between the given cross sections.
Cross sections conveyance is calculated with the specified Manning number and using hydraulic radius (rather than resistance radius).
3. Floodplains.
The bridge opening under the A4104 road has been modelled as a 40 meter wide box culvert using the culvert feature within the 2D component of MIKE FLOOD (MIKE 21).
4. 1D-2D volume transfer
The masonry culvert at the downstream end of Pool Brook has been modeled as a 1D river channel connecting from the 2D model to the river at cross section M033. A 1D culvert controls the flow in the 1D river channel.

Similarly, a 1D channel with a 10 meter wide sluice gate is used to model the flow between flood plain 1 and cross section M030.
Generally, the 1D-2D volume transfer is done with a so-called lateral coupling between the 1D and the 2D model. This implies using a weir equation for the calculating the exchange flow between 1D and 2D models."

5. SOBEK

"Regarding the Test7 model schematization, we like to mention following:
1. The floodplain bathymetry is based on file "Test7DEM_10m.asc", having a
2D grid cell size of 10m.
2. As requested, in breakline 7 the sample point x=385068, y=240140, z=13.211 has been omitted. Breaklines "Test7-Bank-bklines_1 to _7" and "Test7-FPbklines_1 to _8" were transferred into corresponding elevated 2D gridcells.
3. Modelling river flow as 1D flow and its adjacent floodplains as 2D flow, has the disadvantage that the modeller defines the locations, were exchange of 1D river flow and 1D floodplain flow (and visa versa) can occur. These locations are not necessarily the locations were in real-time-situations, exchange of river flow and floodplain flow will occur. Based on good modelling pratice, we recommend to model both the river and its adjacent floodplains as 2D flow. Hence, in this way the location(s) were exchange of river flow and floodplain flow (and visa versa) occurs, depends on the actual governing hydraulic conditions in both the river and its adjacent floodplains. In sections were the river has no adjacent floodplains, we prefer to model the river as 1D flow only, this in order to reduce on required computational time. Taking the above into account, the overall 1D2D model set-up of Test7, comprise of:
a. section with 1D flow only, covering cross-section M013 up to M023,
b. section with 2D flow only, covering cross-sections M023 to M044 as well as floodplain 1, 2 and 3,
c. section with 1D flow only, covering cross-section M044 to M054.
Please note that::
i. The above described three sections a, b and c are run simultaneously. In other words all the St. Venant equations, concerning section a. b and c are solved simulataneously in one and the same matrix. More precisely, sections a and b are internally connected to each other. The same applies for sections b and c.
ii. The river part in section b comprise of a 2D model schematization. The 2D bathymetry of this river section is based on cross-sections M023 to M044.
4. The PoolBrookCulvert (*see Fig 2a and 2b*) is modelled as a 1D pipe, having a length of 28.284 m and linking the river at 2D grid cell (x=384910, y=240900) with floodplain3 at 2D grid cell (x=384890, y=240880). The invert level of the pipe at both the river-side and the floodplain3-side amounts to 6.25m. A Manning value of 0.025 for the 1D pipe, having a diameter of 5000 mm, was applied. Locally near the pipe outflow in floodplain3, 2D bed elevations were lowered to 6.00 m. Furthermore, for some extent 2D gridcells lying under the Pool Brook were locally lowered.
5. An opening (*see Fig 3*) in the embankment (floodplain breakline no 2) was modelled by lowering 2D gridcell (x= x=384610, y=242490) to an elevation of 10 m. Futhermore, locally 2D grid cells were reduced in such way that a some small channel (10m wide) is running from the opening in the embankment toward the local river bed
6. An opening (*see Fig 4*) in road A4104 was made as follows. A 40 m wide opening, with an invert level of 11.40 m was made in both floodplain breakline no 7 as well as in floodplain breakline no 6. Furthermore, locally around the opening some 2D grid cells were lowered."

"In the meeting held on 15-01-2010 it was suggested that Deltares could, if they felt necessary, still supply for test 7 a model in which the entire river is modelled as 1D flow, the floodplains are modelled as 2D flow, and exchange of water from river to flood plain and visa versa is done by means of 1D flow links.

We like to mention that from the SOBEK functionality point-of-view, there are no limitations to construct a model schematization as described above.

However we decided not to submit such a model schematization for test 7, based on Good Modeling Practice.

Deltares nowadays prefers to model a river directly adjacent to the flood plains as 2D flow, meaning that the river as well as its adjacent flood plain are modelled as 2D flow, avoiding arbitrary 1D2D links and human errors in defining the model parameters of the 1D2D links.

Within a SOBEK 2D model schematization no special efforts are needed by the hydraulic engineer to ensure that the correct discharge is flowing over dikes and elevated (rail)roads, since a limiting algorithm automatically ensures this. Correct discharges over dikes means proper exchange of water between river and floodplain and vice versa.

In the meeting we understood that the Environmental Agency has for several rivers quite a number of 1D river schematizations. In respect here with, we like to mention that the incorporation of an existing 1D river model schematization into a SOBEK 2D model schematization, including river as well as adjacent floodplains, is a fast and simple process."

"The construction of a 2D grid on basis of 1D cross-sections, refers to a pre-processing GIS-type of activity (e.g. *Index triangulation*). We offer SOBEK clients the RFGRID and QUICKIN for this purpose. These tools are against additional payment available for any SOBEK user. Actually they form a part of the Deltares Systems modelling suite."

6. TUFLOW

"The conventional hydraulic radius formulation for 1D cross-sections was used so as to be consistent with the other solutions (ie. TUFLOW .ecf file command "Conveyance Calculation == Change in Resistance" was used – refer to the TUFLOW manual). This is not the TUFLOW default which is to carry out a complete parallel channel analysis that treats every segment across the section as a separate parallel channel. This approach ensures that conveyance never decreases with height as can occur with the hydraulic radius approach, but can result in higher conveyance values of around 10%. It is similar to the effect/intent of the resistance radius approach except that side wall friction is taken into account.

The 2D domain sampled elevations from the 1m DEM at the cell centres, mid-sides and corners (ie. a sample grid of 10m resolution).

The A4104 bridge opening was modelled as a causeway using the dimensions specified within the 2D domain.

The DEM elevations at the inlet to the Pool Brook culvert (on the floodplain side) are significantly higher than the culvert invert. A gully line ("Read MI Z Line" command) was created along the natural watercourse leading into the culvert to lower ground elevations near the culvert and direct water into the culvert. The culvert was modelled as specified.

The sluice gate at the southern end of Floodplain 1 was modelled as specified. No modifications to the elevations on the floodplain side were necessary as fortuitously the elevation sample points fell within the open channel leading to the gate.

The results submitted are based on 1D computational network of the same resolution as the frequency of cross-sections. A sensitivity test was carried out with typically two interpolated cross-sections between the provided cross-sections, producing a higher resolution 1D network – this is often needed especially where the 1D longitudinal water level profile is not linear (eg. around a meander). For this model, using a higher resolution 1D network had only a very minor effect on flood levels in both 1D and 2D domains."

APPENDIX D – FloodFlow results

1. Background

Micro Drainage provided results from simulations using a depth-dependent roughness approach (see Appendix B.1 for more detailed information), as follows:

Depth (m)	Manning's n
0.1	0.1
0.25	0.03
0.5	0.012
1	0.01

These results were therefore not directly comparable to others, and are presented separately in this appendix. Additional results were provided for Test 1 only based on the specified constant roughness value. This is commented on in Section 4.1 and presented again below.

2. Comments

It can be assumed from the information above that Manning's n values effectively used in the FloodFlow models compared to the specified values (see Appendix A) as follows:

- In tests 1 and 2 they were much larger in shallow areas between the depressions, comparable elsewhere;
- In test 4 they varied with distance (from the inflow) from much smaller to larger;
- In test 5 they were usually much smaller;
- In test 7 they were often much larger along river banks, otherwise much smaller;
- In test 8 they were usually much larger;

Results from the FloodFlow simulations are compared in Sections 3 to 9 below to results by two unnamed packages based on the full shallow water equations. The differences observed are consistent with the use of different Manning's n values in the FloodFlow model: lower values usually result in larger velocities, shorter travel times and smaller depths, while higher values usually result in smaller velocities, longer travel times and larger depths.

Oscillatory solutions are observed, particularly of velocities for tests 4, 5 and 7, with often an initial sharp peak in the velocity prediction. This may be due to the use of lower Manning's n values and to numerical difficulties in the modelling of the resulting fast, possibly supercritical, flows.

The following comments can also be made:

Test 1: the results using a constant Manning's n are consistent with those of other packages.

Test 2: the flood has not reached its final state at the end of simulation, but the distribution of inundation would be expected to be different from that predicted by other models, as inertia and friction were important processes in this test.

Test 4: a rapid decrease in levels and velocities at the end of the event is observed, likely due to a misapplication of the specified inflow boundary condition.

Test 7: river levels, compared to other models, where ~0.5m higher at the north end to ~0.2m lower at the south end, raising questions about FloodFlow's robustness when applied to the modelling of rivers[37]. The behaviour predicted in the floodplains is otherwise consistent with the predicted river levels: higher peaks in Floodplain 1 and 2, and very reduced flooding in Floodplain 3 (downstream flooding was also probably affected by the fact that the floodplains to the north were flooded to larger depths).

Final levels predicted, which are governed by the elevations of low points along the banks or embankments (although not in floodplain 3 as it did not fill up in the FloodFlow model) are not accurately predicted, at point 1 by almost a metre, at points 2 to 6 by ~0.2m, at points 7 to 9 by ~0.1m to ~0.2m.

The specified structures (sluice at south end of floodplain 1, culvert at north end of floodplain 3) were not included in the FloodFlow model.

Test 8A: The first peak (due to rainfall) observed with other models is not visible at all some locations (see points 1,2,4,7). This could be due to the very large values of Manning's n used in areas of very shallow flow. (The final levels predicted at points 3 and 5 (large downstream pond) also suggest a very significant deficit in water volume at the end of the computation.)

Conclusion:

The depth-dependent approach to roughness used in FloodFlow (see Appendix B.1) is not commented on. However the fact that this approach was used in this benchmarking exercise limits any opportunity to draw conclusions from FloodFlow's results. The flows predicted are significantly different from those predicted by other models, and it is impossible to apply to FloodFlow the level of scrutiny that is applied in Section 4 in the analysis of other models.

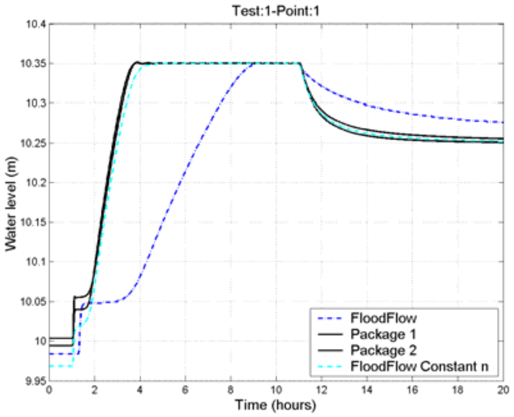
It is observed that wherever lower than specified (and also than usual engineering practice) Manning's n values were used, FloodFlow's simulations were prone to numerical oscillations.

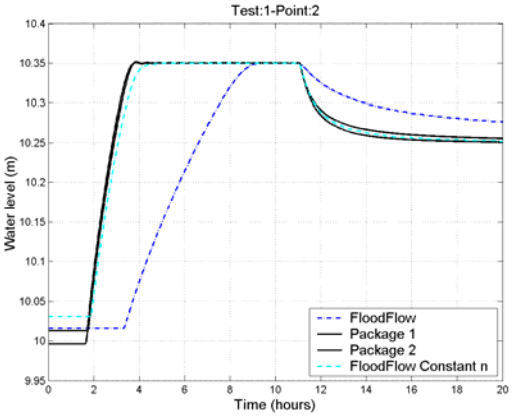
The analysis above also suggest limitations in the application of FloodFlow to combined river and floodplain systems (Test 7).

It is however acknowledged that in the case of Tests 1,2,4,5 and 8 the results do not seem inconsistent with expectations considering the values of Manning's n used (this comment is qualitative).

[37] As commented on by MicroDrainage: "As well as the effects of Manning's n we also believe the differences in the answers are caused by limitations within the 1D engine when analysing such large open sections. FloodFlow was developed principally to analyse very shallow pluvial flooding within an urban development rather than being aimed at full river catchment modelling."

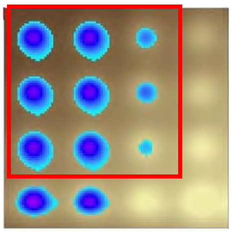
3. Results from Test 1

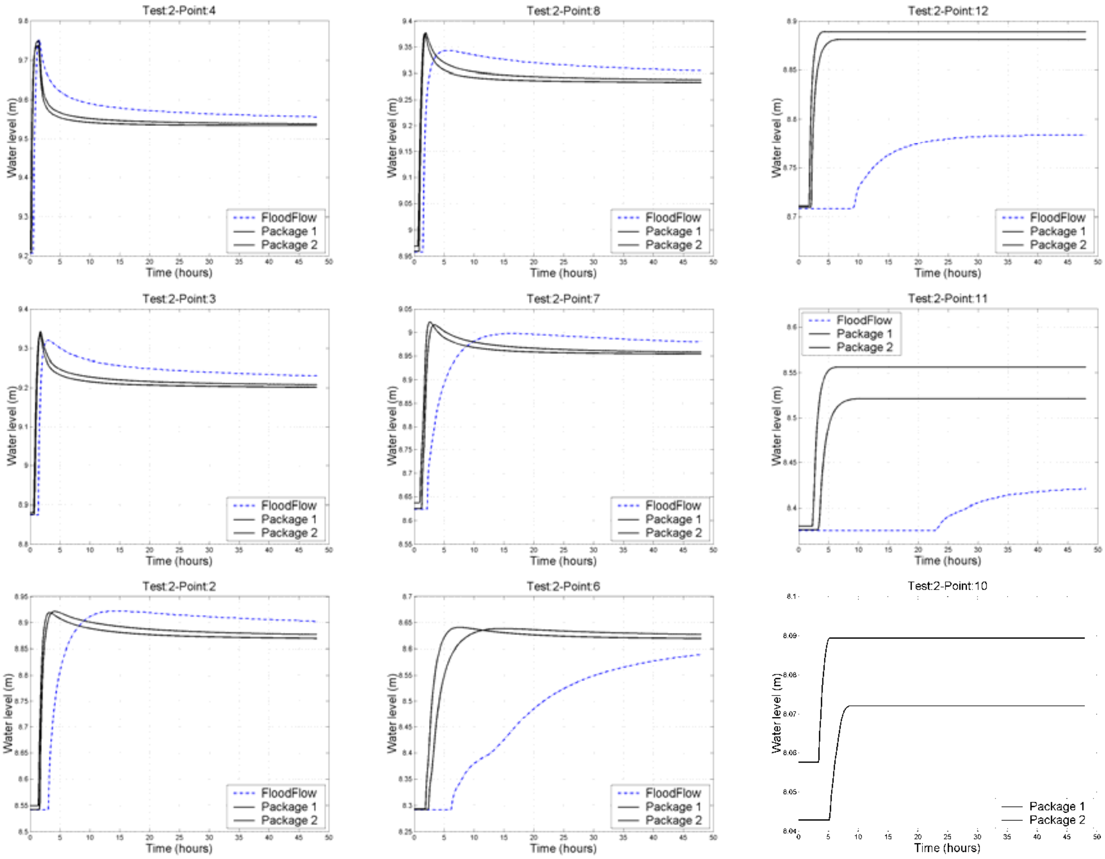


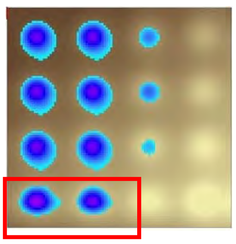


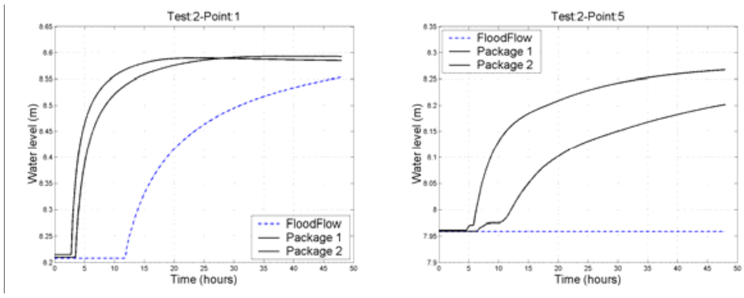
4. Results from Test 2

FloodFlow predicted point 10 to remain dry, as well as points not shown.

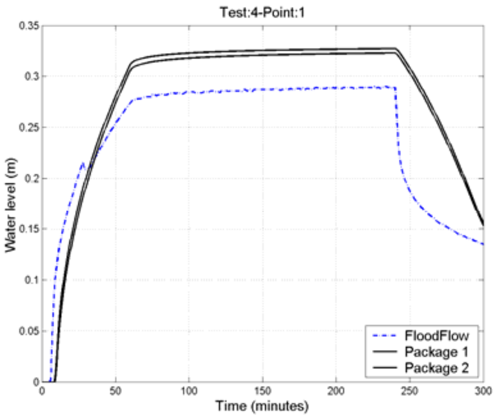








5. Results from Test 4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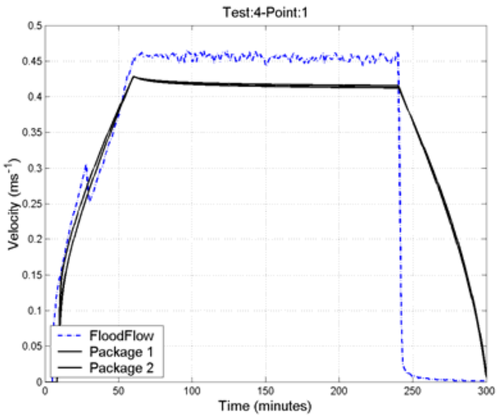
Water level (m)
Time (minutes)
FloodFlow
Package 1
Package 2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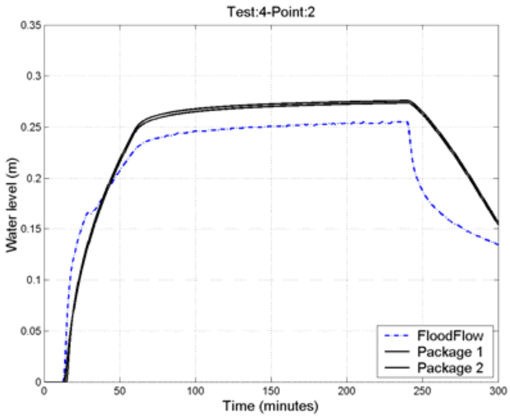
Velocity (ms^{-1})
Time (minutes)
FloodFlow
Package 1
Package 2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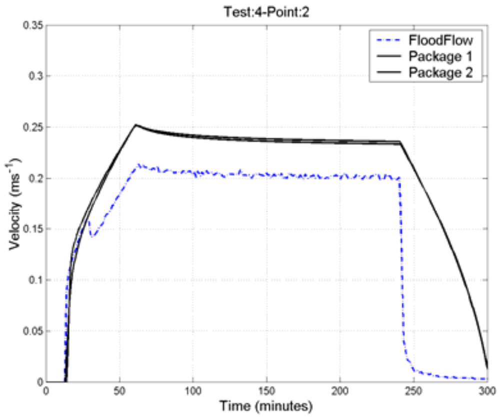
Time (minutes)
FloodFlow
Package 1
Package 2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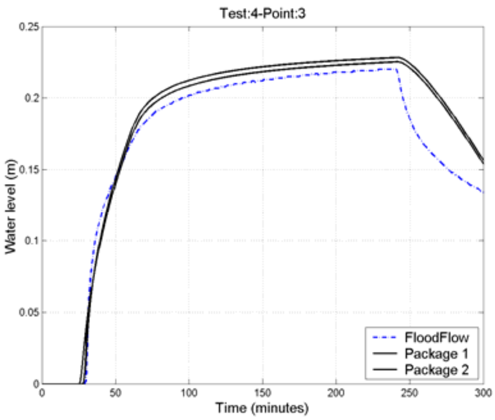
Time (minutes)
FloodFlow
Package 1
Package 2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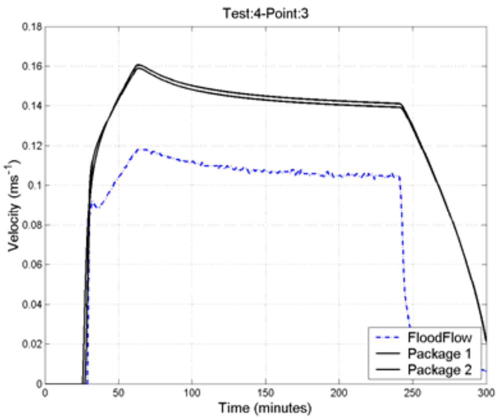
Water level (m)
Time (minutes)
FloodFlow
Package 1
Package 2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Time (minutes)
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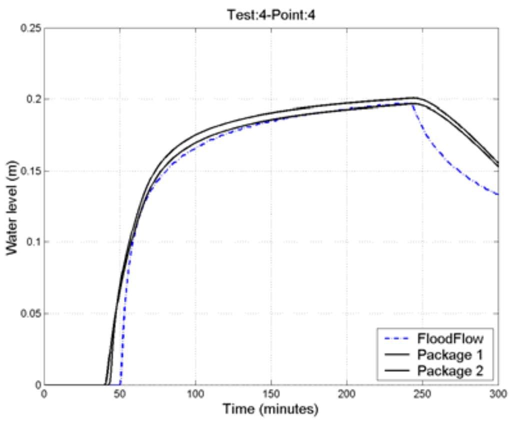
Package 2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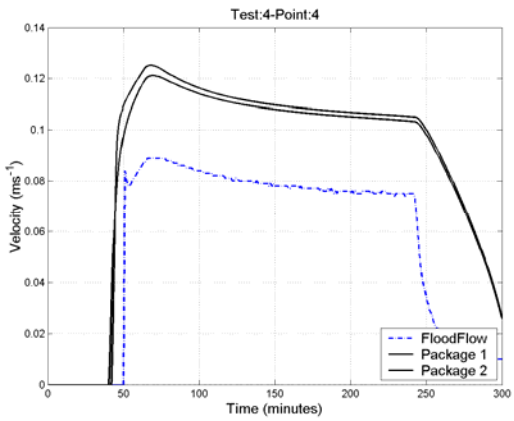
Water level (m)
Time (minutes)
FloodFlow
Package 1
Package 2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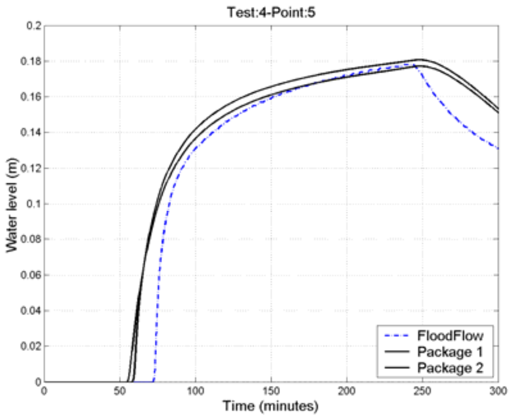
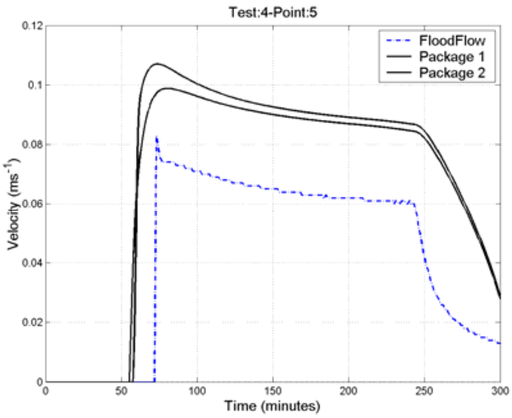
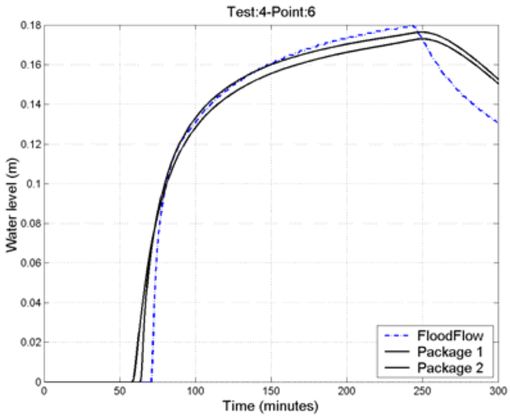
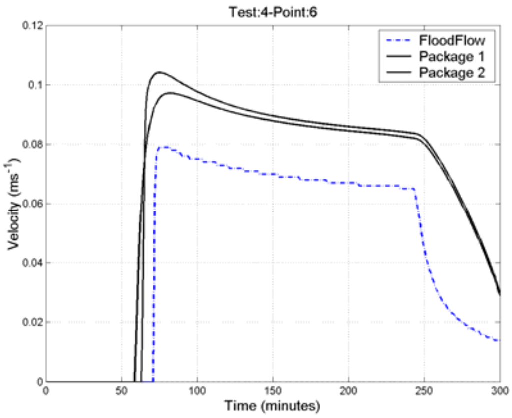
Velocity (ms^{-1})
Time (minutes)
FloodFlow
Package 1
Package 2

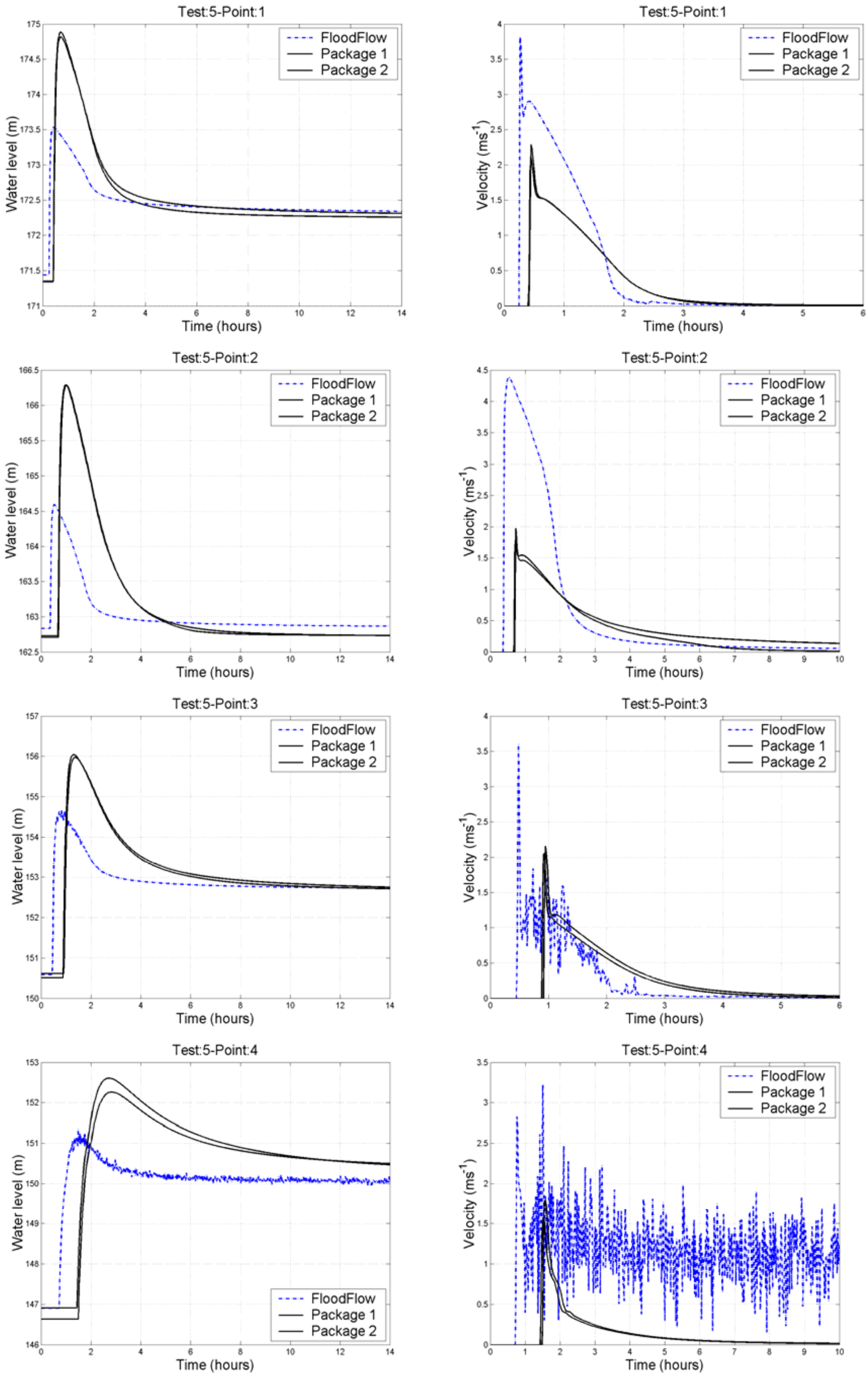


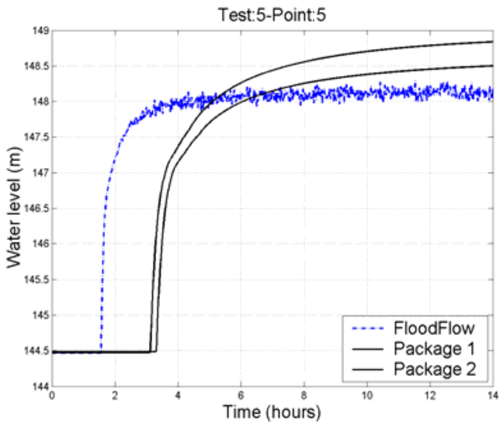


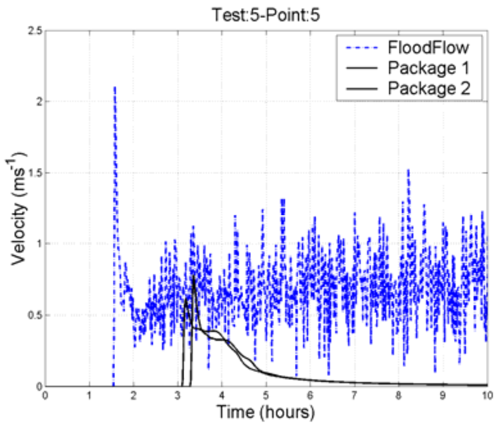




6. Results from Test 5


Test:5-Point:1
Water level (m)
Time (hours)
FloodFlow
Package 1
Package 2
Test:5-Point:1
Velocity (ms-1)
Time (hours)
FloodFlow
Package 1
Package 2
Test:5-Point:2
Water level (m)
Time (hours)
FloodFlow
Package 1
Package 2
Test:5-Point:2
Velocity (ms-1)
Time (hours)
FloodFlow
Package 1
Package 2
Test:5-Point:3
Water level (m)
Time (hours)
FloodFlow
Package 1
Package 2
Test:5-Point:3
Velocity (ms-1)
Time (hours)
FloodFlow
Package 1
Package 2
Test:5-Point:4
Water level (m)
Time (hours)
FloodFlow
Package 1
Package 2
Test:5-Point:4
Velocity (ms-1)
Time (hours)
FloodFlow
Package 1
Package 2


Test:5-Point:5
Water level (m)
Time (hours)
FloodFlow
Package 1
Package 2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Time (hours)
FloodFlow
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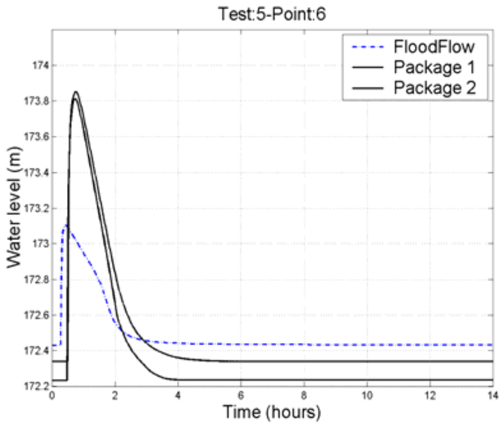
Package 2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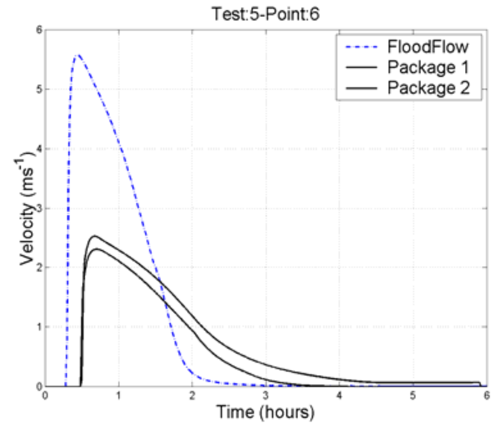
Water level (m)
Time (hours)
FloodFlow
Package 1
Package 2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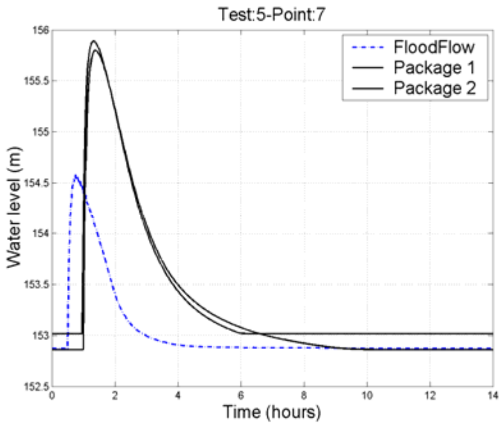
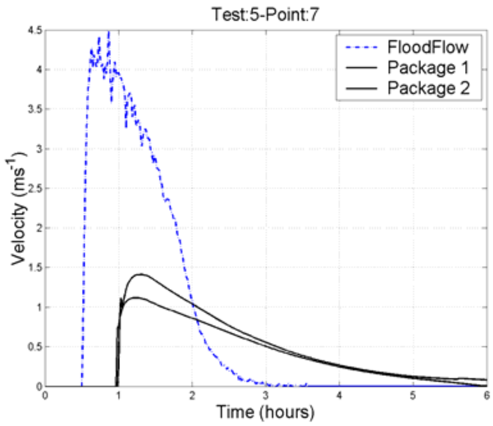
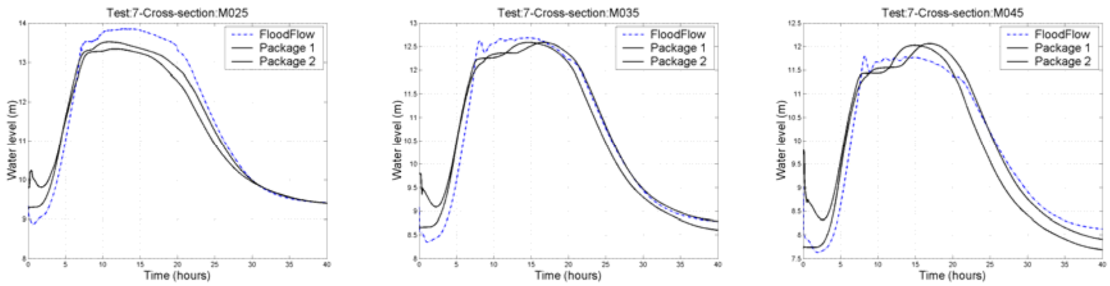
Velocity (ms-1)
Time (hours)
FloodFlow
Package 1
Package 2


Test:5-Point:7
Water level (m)
Time (hours)
FloodFlow
Package 1
Package 2


Test:5-Point:7
Velocity (ms-1)
Time (hours)
FloodFlow
Package 1
Package 2

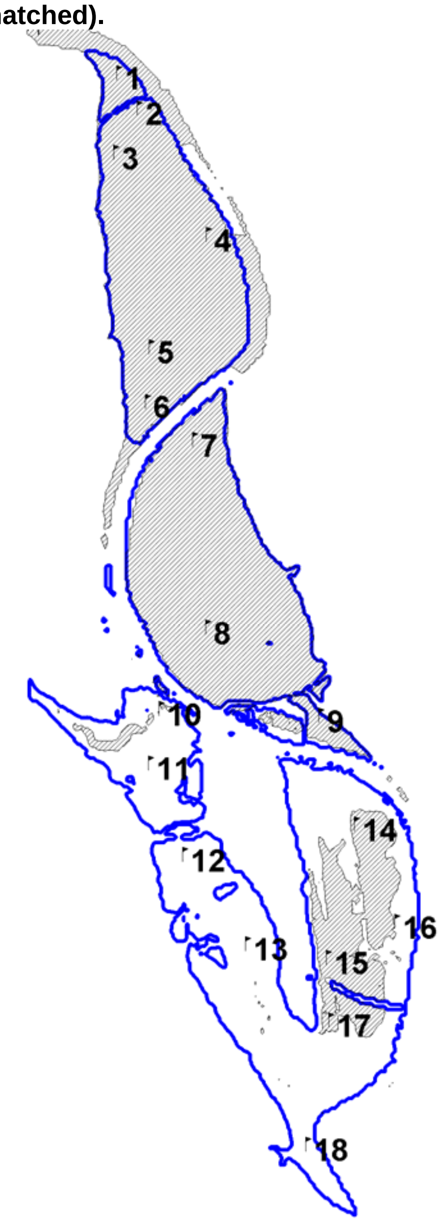
7. Test 7

River water levels



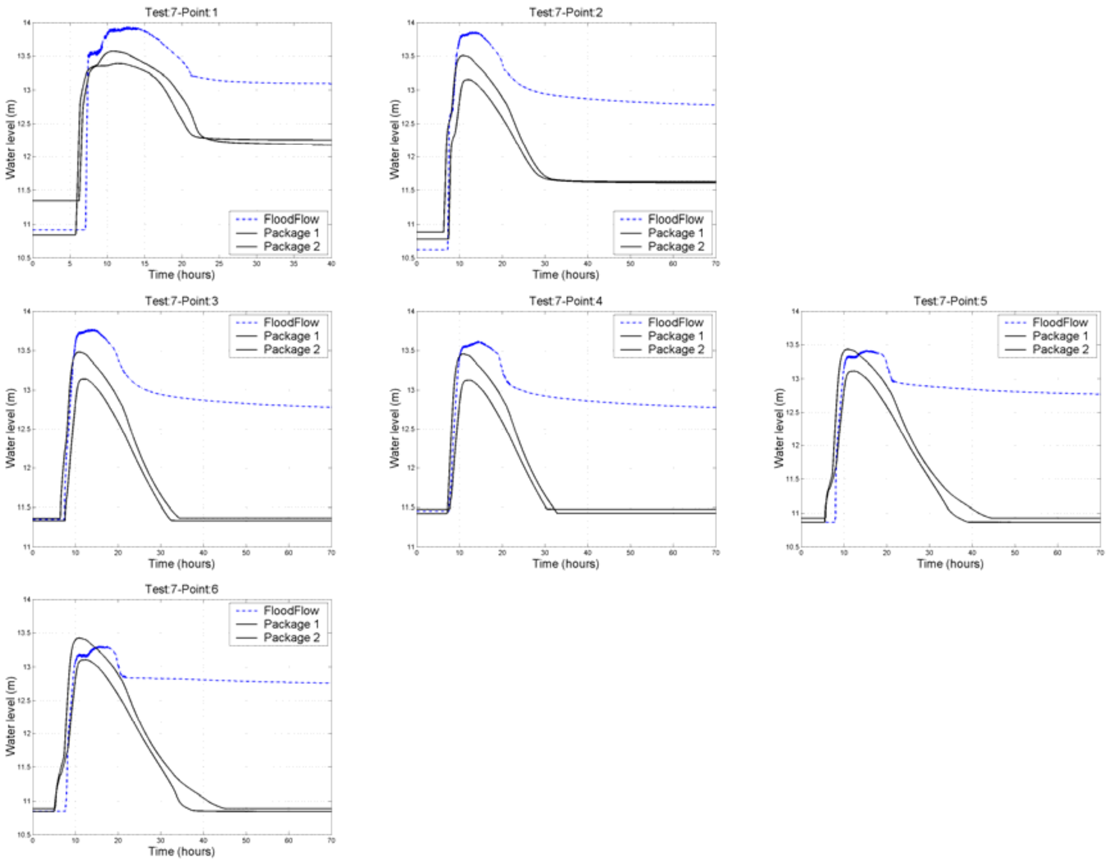
Peak inundation extent

Figure 22: 50cm contour line of peak depth as predicted by MIKE FLOOD (blue) and FloodFlow (hatched).

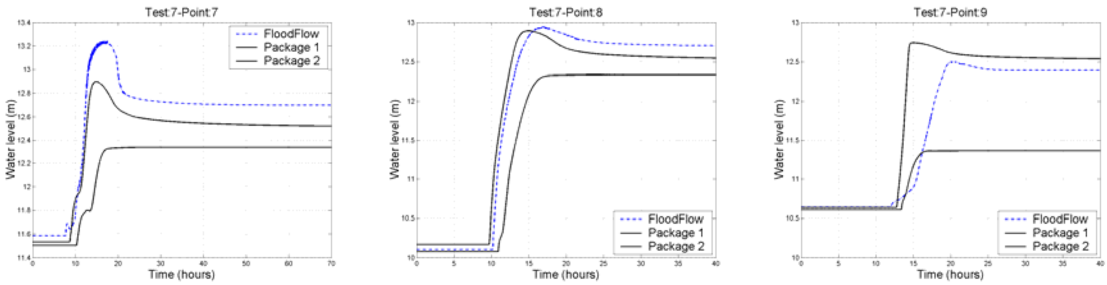


Floodplain Water Levels

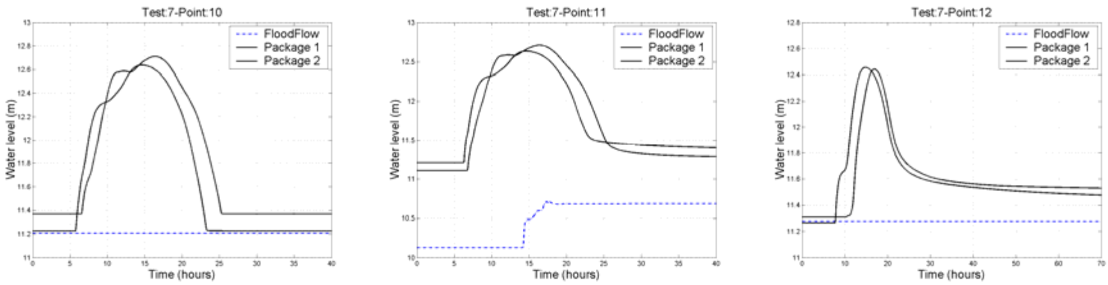
Floodplain 1

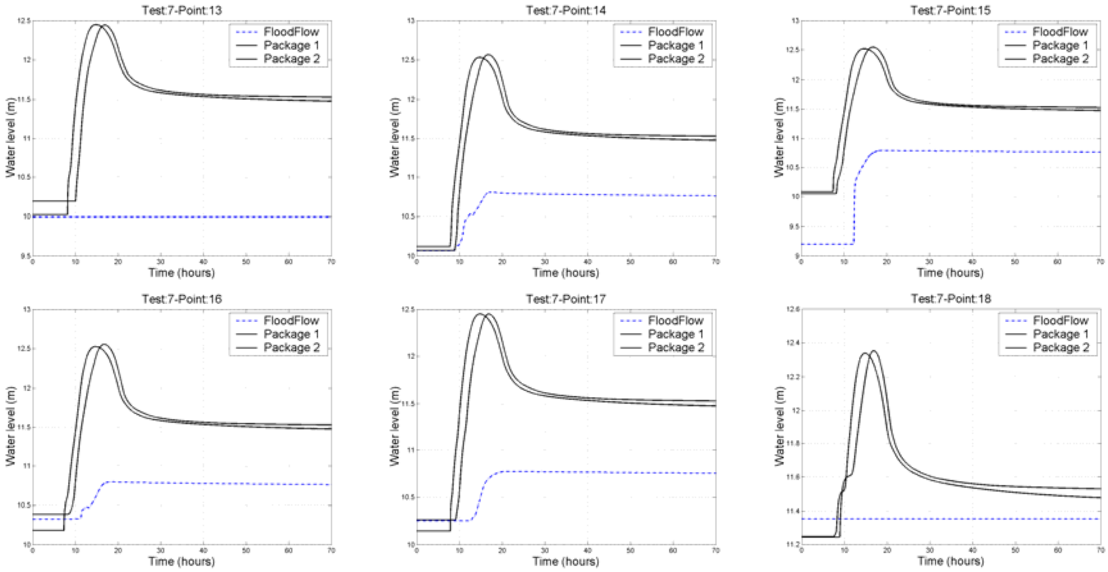


Floodplain 2

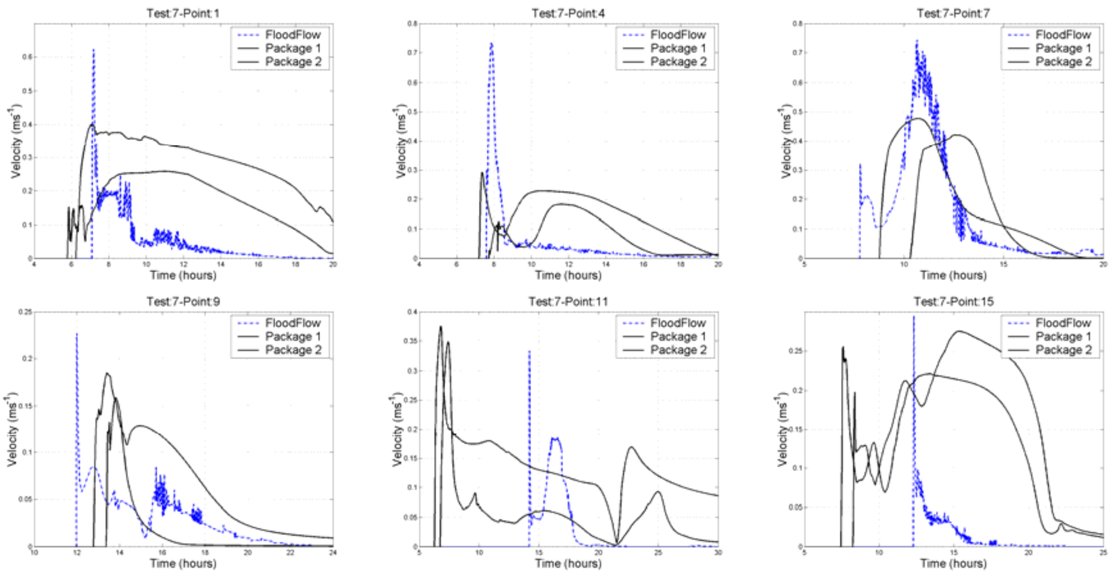


Floodplain 3

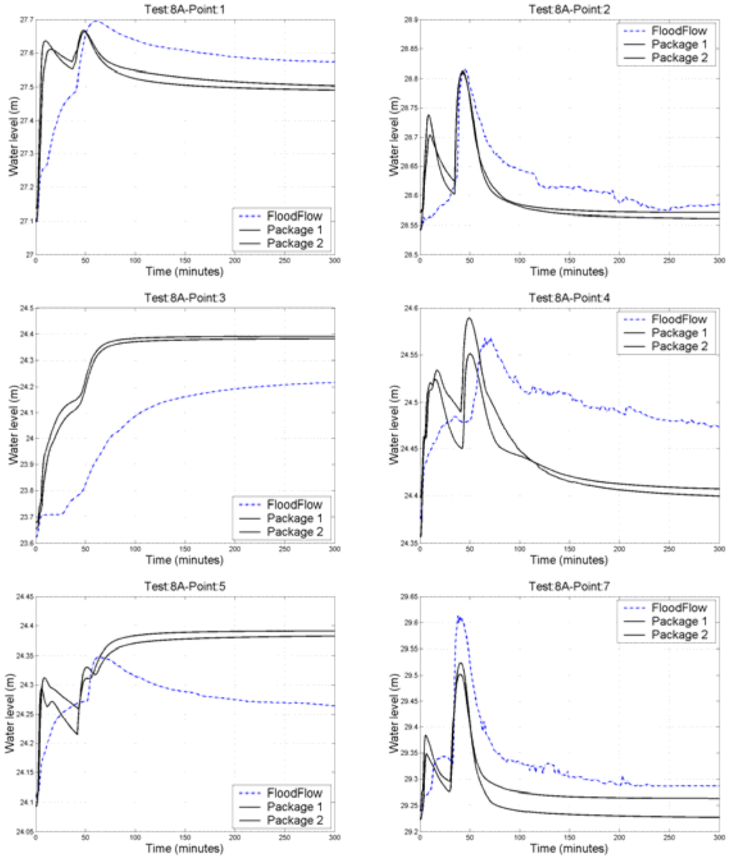




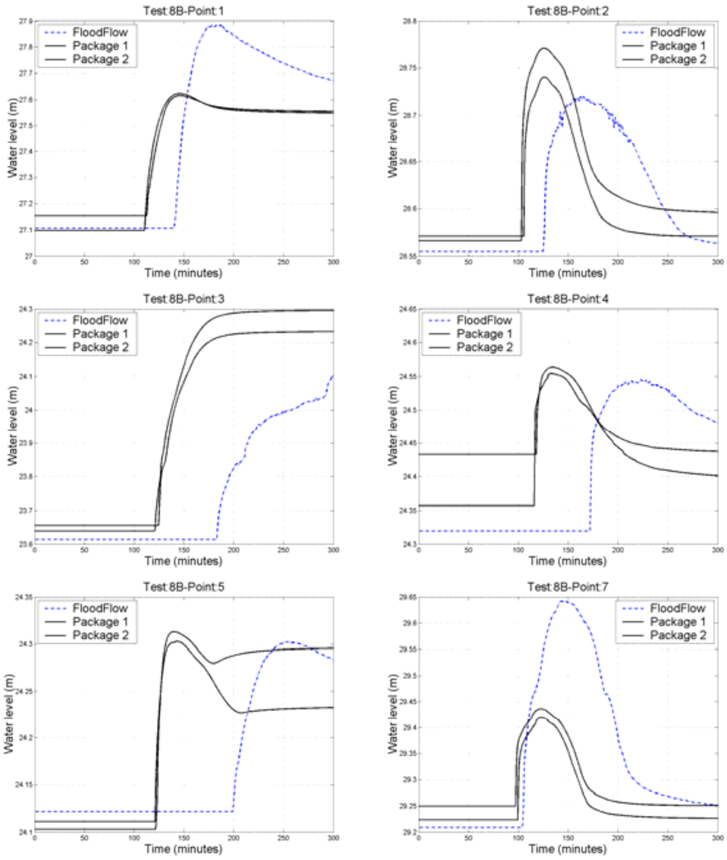
Floodplain Velocities



8. Test 8A


Test:8A-Point:1
Test:8A-Point:2
Test:8A-Point:3
Test:8A-Point:4
Test:8A-Point:5
Test:8A-Point:7
FloodFlow
Package 1
Package 2
Water level (m)
Time (minutes)

9. Test 8B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Test:8B-Point:2
Test:8B-Point:3
Test:8B-Point:4
Test:8B-Point:5
Test:8B-Point:7
FloodFlow
Package 1
Package 2
Water level (m)
Time (minutes)

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