Defra/Environment Agency
Flood and Coastal Defence R&D Programme

Benchmarking Hydraulic River Modelling Software Packages

Results – L (Contraction & Expansion)

R&D Technical Report: W5-105/TR2L

TECHNICAL REPORT W5-105/TR2L

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R&D Technical Report: W5-105/TR2L

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Statement of use
This document provides the results and findings from undertaking the Environment Agency's Benchmarking Test L (Contraction and Expansion) for hydraulic river modelling software. The results only relate to the ISIS, MIKE 11 and HEC-RAS software packages and inference to the likely performance to other software packages should not be made.

The findings are intended to be a supplementary resource for Defra and Agency staff, research contractors and consultants, academics and students for assessing the applicability of any one of these software packages for their own modelling requirements. This report should not be considered in isolation and should be read in conjunction with the other tests reports produced as part of this R&D project.

Keywords
Hydraulic Modelling, River Modelling, Benchmarking, Test Specifications, Contraction and Expansion, Dam Failure

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EXECUTIVE SUMMARY

The test is an extreme investigation of the capabilities of the software packages due to the nature of the hydraulic condition and the scale of the study. As a result several notable nuances between the software packages have been highlighted.

Due to the small time step required in order to obtain a solution for the test HEC-RAS has returned a “no result” for the test, as it is limited to a minimum time step of 1 second. However, it is acknowledged that HEC-RAS may well be able to model this situation if more ‘real life’ prototype scaling had been used.

The results from ISIS are inconclusive due to notable instabilities in the calculated water levels; however, the final calculated depth of water produced by ISIS at the downstream measuring station is exactly equal to that produced in MIKE 11.

In comparison to ISIS, MIKE 11 has been shown to be more flexible in the choice of initial conditions and in the value for the implicit weighting factor used in the numerical scheme. This may suggest that the numerical scheme adopted by MIKE 11 may be more robust than that employed by ISIS for conditions where there are significant/extreme velocity or water level changes.

Only MIKE 11 was able to produce stable solutions for the dam-break progressive wave; however, the results calculated by this solution were not truly representative of the problem. The calculated water levels for MIKE 11 show a delay in the arrival time of the initial surge wave, which was also an observation of the results of the two-dimensional models used to model the same dam-break in the original CADAM project.

The test has not extensively attempted to improve on model accuracy through the refinement of the calculation settings and options. Hence, it is anticipated that the software packages are capable of producing results that have greater accuracy and stability then presented herein. It is recommended that any future testing should attempt to find the most appropriate and improved settings for this test and tests of this nature.

The modelling approach required for this test has not provided a true real life test of the software packages, hence it is recommended that the test specification be enhanced and added to so as to consider a more real-life prototype situation.

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1 INTRODUCTION

1.1 Background

This report presents the results and findings from Test L (Contraction and Expansion) of the Environment Agency of England and Wales (EA), Benchmarking and Scoping Study (2004). The study, which encompasses a series of tests, is intended to be an independent research investigation into the accuracy, capability and suitability of the following one-dimensional hydraulic river modelling software packages:

Software	Version		Developer
ISIS	User Interface:	2.0 (13/01/01)	Halcrow /
	Flow Engine:	5.0.1 (27/06/01)	Wallingford Software
MIKE 11	User Interface:	Build 5-052 (2001b)	DHI Water and Environment
	Flow Engine:	5.0.5.5	
HEC-RAS	User Interface:	3.1.0 (Beta) (03/02)	US Corps of Engineers
	Pre-processor:	3.1.0 (Beta) (03/02)	
	Steady Flow Engine:	3.1.0 (Beta) (03/02)	
	Unsteady Flow Engine:	3.1.0 (Beta) (03/02)	
	Post-processor:	3.1.0 (Beta) (03/02)	

Each of the above software packages was tested in the previously undertaken benchmarking study (Crowder *et al*, 1997). They are currently on the EA's BIS-A list of software packages for one-dimensional hydraulic river modelling.

The test has been undertaken on behalf of the EA by the following team in accordance with the Benchmarking Test Specification - Test L (Contraction and Expansion), (Crowder *et al,* 2004):

	Role	Affiliation
Mr Andrew Pepper	EA Project Manager	ATPEC River Engineering
Dr Richard Crowder	Study Project Manager	Bullen Consultants Ltd
Dr Nigel Wright	Advisor	University of Nottingham
Dr Chris Whitlow	Advisor	Eden Vale Modelling Services
Dr Andrew Sleigh	Advisor	University of Leeds
Dr Chris Tomlin	Advisor	Environment Agency
Mr D Cross	Tester/Reporter	University of Leeds

1.2 Aim of Test

The aim of the test is to:

- assess the ability of the software packages to replicate the behaviour of a surge wave, caused by the sudden collapse of a large body of water, in a channel with a local constriction and expansion;

- benchmark the numerical results against laboratory results obtained in the European Commission's CADAM project – the European Concerted Action on Dam-Break Modelling (Soares and Alcrudo , 1998); and

- present the particulars for developing and undertaking the test (Model Build) with each of the software packages and the associated results so that others can repeat the test with their own software.

1.3 The CADAM Project

The CADAM project was set up and funded by the European Commission in 1998, as a two year programme of work performed by participants from over ten different European countries, to further computational modelling techniques into dam-break analysis.

Due to the very irregular and unpredictable nature of dam-break surge flows, benchmark data was gathered based upon laboratory models designed to recreate a dam-break in simple channels. The physical models were used as a means by which to evaluate the quality of the hydraulic flow data generated using numerical methods and computational models.

Three test cases were proposed in the CADAM report 3; Test Case 1 – L Shaped Channel; Test Case 2 – Local Constriction; and Test Case 3 – Flood Plain. All three tests were performed in channels of uniform rectangular cross-section, and were designed to provide simplified results for a surge wave experiencing:

- a change in direction downstream of the reservoir;
- a constriction in the flow; and
- an expansion of the flow area respectively.

The results presented within this investigation are based on Test Case 2 for a Dam-Break in a Channel with a Local Constriction

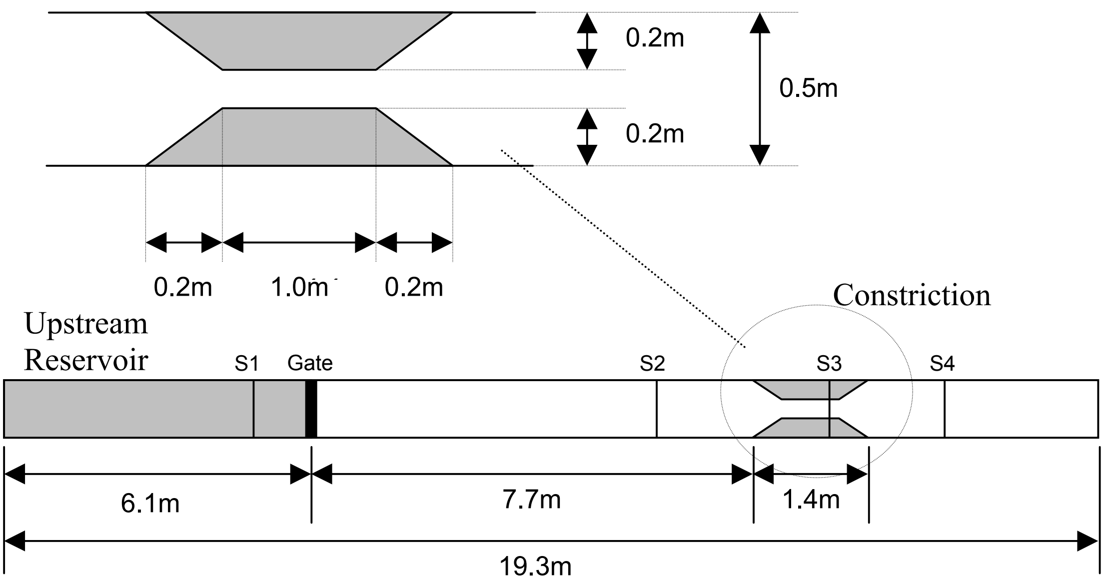
To enable the future comparison with numerical results, water levels, flow velocities, and discharge were measured using staging meters and pressure gauges at specific channel locations.

1.4 The CADAM Laboratory Configuration

The physical laboratory model for the 'Dam-Break in a Channel with a Local Constriction' was built, and the test performed, at the 'Laboratório Nacional de Engenharia Civil' in conjunction with the 'Instituto Superior Técnico' in Portugal 3. A schematic diagram of the channel is given in Figure 1.1.

The model comprises a horizontal 0.5m wide channel of uniform rectangular cross-section. The overall length of the channel was set at 19.30m, with the first 6.10m of the channel at the upstream end specified as the reservoir. A removable sluice gate was built into the channel to retain the water within the reservoir, the gate being removed in approximately 0.2s to simulate the break of the dam. A constriction was located 7.70m downstream of the sluice gate. The constriction was given an overall length of 1.4m, the first and last 0.2m of which were tapered at 45 degree angles to the channel walls. The middle width of the constriction of 0.1m therefore remains uniform for 1.0m in length, as illustrated in Figure 1.1.

Figure 1.1 Schematic Diagram of Channel with Local Constriction (m)



The initial conditions for the test were set at 0.3m depth of water in the reservoir upstream from the gate, and 0.003m in the channel downstream from the gate. The experimental data for the test was obtained by removing the gate, and measuring the depth and velocity of flow at the benchmarking stations S1 to S4 (Figure 1.1). S1 is located 1.0m upstream of the dam gate, S2, 6.10m downstream of the dam gate, S3, 8.80m downstream of the dam gate, and S4, 10.50m downstream of the dam gate. The measurements were taken every 0.04s and recorded up to a simulation time of 10.00s.

2 MODEL BUILD

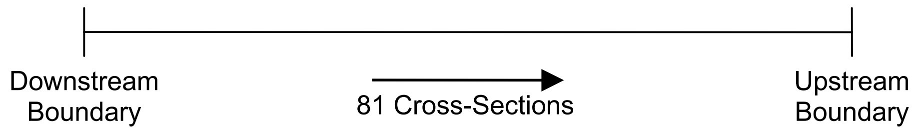
2.1 Test Configuration

In order to be able to recreate the same model in the three software packages a number of different options were considered.

Initially, the construction of a model that exactly replicated that of the physical model was considered, i.e. use a sluice gate unit within the software packages to simulate the gate and its operation. However, each of the software packages model sluice gates in a slightly different way. Hence, it was decided for this study that the model should be set up without a controlling gate and that initial water levels and flows should be specified at every section along the channel at some specific time after the removal of the sluice gate. By setting up the model in this way any stability/operational problems (numerical) that may have been caused by the operation of the sluice structure would be negated.

The configuration for the test is illustrated schematically in Figure 2.1.

Figure 2.1: Schematic Illustration of Test Configuration



The reservoir was extended in length to 26.8m, chosen to give an arbitrary length of channel of 40m. This value was chosen as a precautionary measure to ensure the reservoir would be sufficiently long that there would be no change in water level at the extreme upstream boundary after 10s of the simulation. The recorded values are not affected by extending the tank as they are downstream and the flow is supercritical.

The resulting channel was built up from 81 equally spaced cross-sections of 0.5m spacing with a further 5 used to specify the constriction, and an additional 3 to specify the location of the data benchmarking stations S1, S2, and S3. Benchmarking station S4, located 2.5m upstream of the downstream end of the channel, coincided with one of the 81 equally spaced sections. A flow/time boundary of constant zero discharge was specified at the upstream end of the channel, and a continuous head/time boundary at the downstream end of 0.003m

A constant Manning’s n value of 0.010 has been used throughout the reach.

2.2 Initial Conditions

Applying initial conditions to the model enables water levels, velocities and discharge to be set within the channel prior to commencement of the unsteady simulation. To determine the appropriate initial conditions for the test a computer program was written to obtain an analytical solution of the dam-break problem using the exact Riemann solver (Toro, 1992).

This analytical solution provides the initial stages of a developing surge wave in any given channel, with any specified number of cross-sections, and up to any chosen stage in the development of the surge wave. In this case, three stages of development for the initial conditions were chosen to determine which produced the best results for the unsteady run. The following three developments were tested:

Time = 0.00s – No wave development
Time = 1.00s – Partial wave development
Time = 3.35s – Full wave development prior to the constriction

The analytical solution at the above three times is illustrated in Figures 2.2, 2.3 and 2.4.

Initial simulations were performed using a range of time-steps (0.01s to 0.1s) with the simulation time being approximately 10s.

The initial simulations of the test, based on the above three initial conditions, found that ISIS was unable to produce any stable water levels for any time-step employed, with the software failing to complete the majority of the simulations attempted. Convergence problems were indicated by the software, often resulting in divergence of the solution parameters. When the simulation was undertaken using the initial conditions at time = 0.00s (no wave development), the software could only generate water levels over the entire length of the channel equal to the downstream depth of 0.003m.

Due to the very low time-step required for the dam-break model, it was discovered that this simulation could not be performed in HEC-RAS and as such HEC-RAS has produced a "no result" for the test.

HEC-RAS is configured such that the modeller is restricted to a minimum time-step of 1.0 second, and a minimum save interval to record the data, of 1.0 minute. As the surge wave reaches the constriction in less than 5s no useful results could be generated in HEC-RAS.

In comparison to ISIS and HEC-RAS, solutions for the surge wave were generated in MIKE 11 when using all three of the initial conditions. However, it must be noted that the most stable result was produced when using the initial conditions for a partly developed wave. When using the initial condition at 0.00s a very irregular water surface profile was produced in the early stages of the simulation, as can be seen in Figure 2.5.

Figure 2.2 Initial water levels at time = 0.00s (from analytical solution)

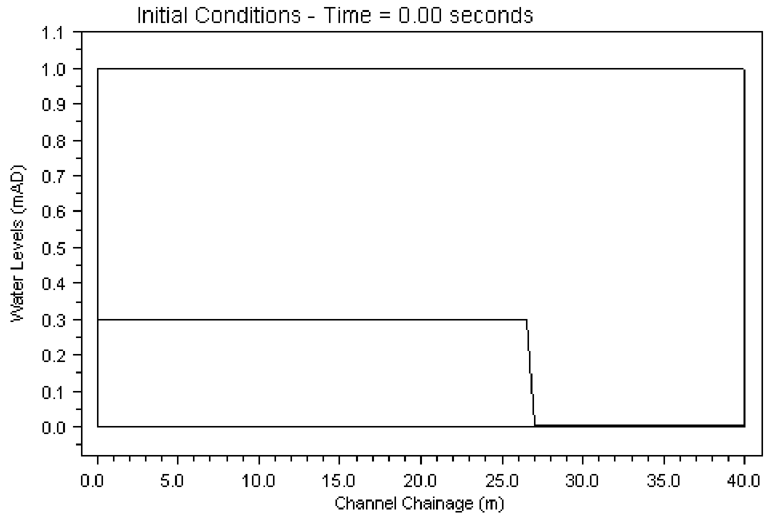


Figure 2.3 Initial water levels at time = 1.00s (from analytical solution)

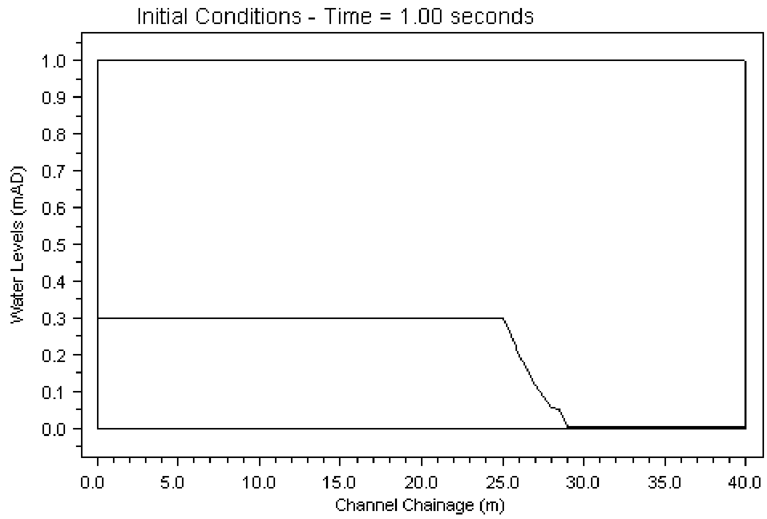


Figure 2.4 Initial water levels at time = 3.35s (from analytical solution)

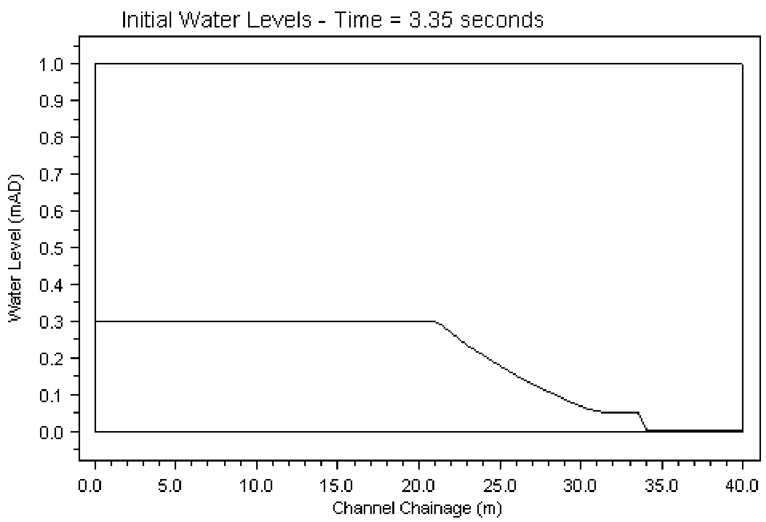
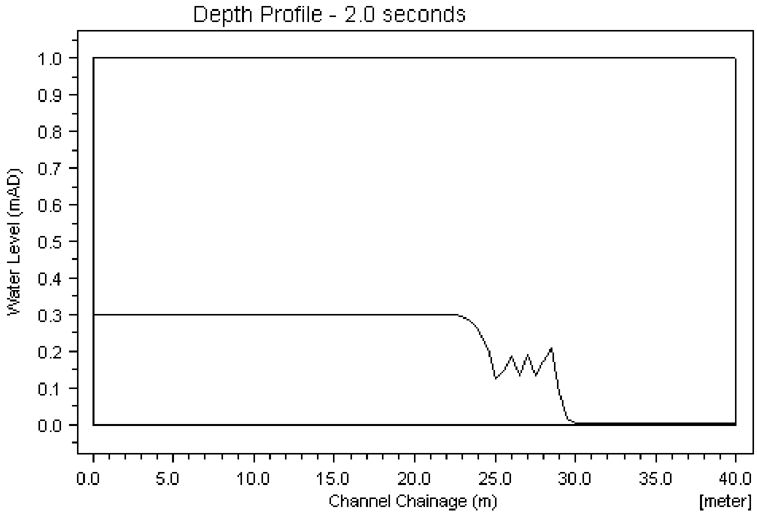


Figure 2.5 MIKE 11 water levels at 2.0s (from initial conditions for no wave development)



As the most stable solutions for the surge wave were generated using MIKE 11, it was decided to use the results of the initial simulations performed in this software as a guide to select the final test configuration. It was therefore decided to begin the simulation from the partially developed wave profile given in Figure 2.3, with the run time of the simulation set at 9s.

3 RUNNING THE MODEL

3.1 Introduction

Further to the investigations with the initial conditions each simulation was started from the partially developed wave profile given in Figure 2.3, with the run time of the simulation set at 9s and the time-step set to 0.04s. Results were recorded at every time-step to be consistent with the results of the original laboratory test.

When running the simulations in both ISIS and MIKE 11 it was necessary to improve the stability of the results by increasing the implicit weighting factor values (θ and δ for ISIS and MIKE 11 respectively), in the advanced options of the simulation editors.

If there is instability in any given model it is generally advised to improve the model geometry and/or initial conditions. However, as the tests performed within this study are designed to draw comparisons between the three different software packages the specifications of the test were kept consistent. As such, the initial conditions and geometry of the models have remained unaltered, but θ and δ were changed accordingly.

3.2 Running the Model in ISIS

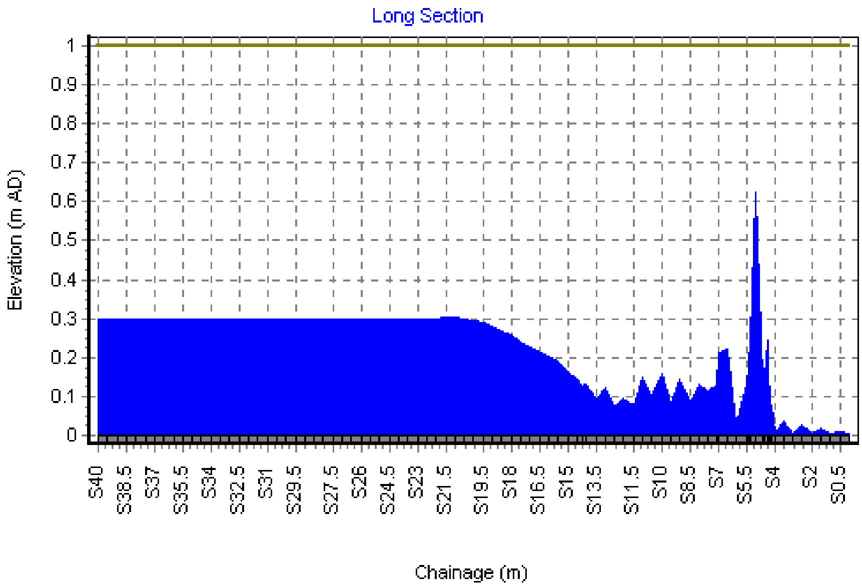
When the unsteady simulation was performed in ISIS, it was found that the time-step and save interval of 0.04s could not be specified simultaneously. This was partly due to the fact that the simulation time can only be entered in hours, meaning that a decimal value of 0.0025 had to be entered to correspond with a simulation time of 9s. Warning messages were produced by the software, stating that the specified simulation time could not be used with the given time-step, and that the time should be reset to 0.002489, equivalent to 8.9604s. This was tried, but the simulation diverged.

The simulation was then performed with the same time-step and save-interval, but for 10s or 0.00278hrs. A similar warning was again given, suggesting the time be reset to 0.002767hrs, equivalent to 9.9612s. Again the solution diverged. It was then considered whether a solution could be found by increasing the save-interval. This was therefore changed, and a value of 0.12s was chosen arbitrarily as a multiple of 0.04, i.e. the data would be saved every 3 time-steps.

It was found that ISIS could perform the test without the solution diverging if a time-step of 0.06s, and a save-interval of 0.12s was used.

It was found that Theta 'θ' had to be increased to reduce early instabilities in the model. The best solution was achieved using $\theta = 1.0$. However, it must be noted that the software indicated poor model convergence for the first 3.6s of the simulation. A typical graphic from ISIS showing the resultant water surface profile is given in Figure 3.1.

Figure 3.1   Water surface profile in ISIS at approximately 3.0s

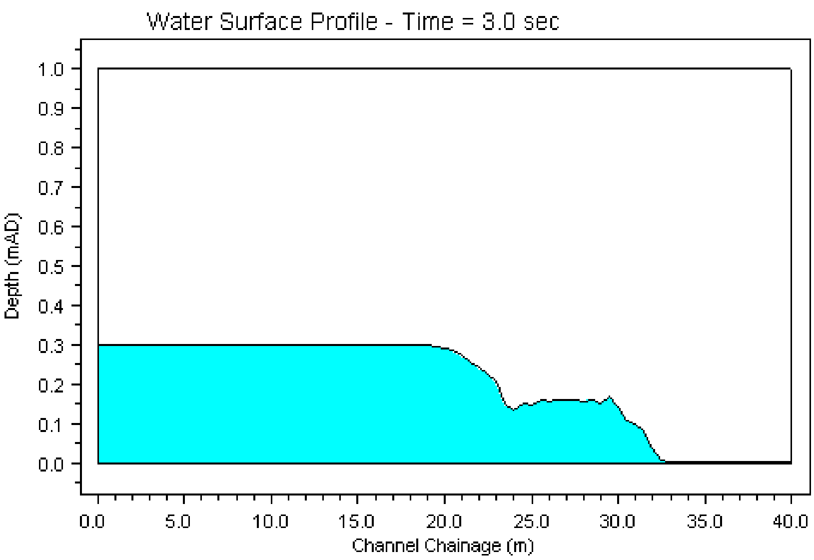


3.3   Running the Model in MIKE 11

Performing the simulation in MIKE 11 was far more successful. Despite the fact that many of the solutions generated in the initial testing of the model showed relatively unstable results every simulation tried could be performed without the onset of divergence problems as experienced in ISIS.

The simulation was run successfully according to the specifications for the model. However, some instability was noted in the water surface profile when the model was run using a delta ‘δ’ value of 0.5. This was more than adequately smoothed by increasing δ to 0.65. It can be seen from Figure 3.2, a typical MIKE 11 graphic, that the resultant water surface profile generated in MIKE 11 is significantly smoother than that produced in ISIS after the same amount of time.

Figure 3.2   Water surface profile in MIKE 11 at approximately 3.0s



4 RESULTS

4.1 Introduction

The analysis of results from each software package has been limited to the following:

Stage versus Time at S1, S2, S3 and S4.

An alternative test has also been undertaken with MIKE 11 using a Chézy C value of 70.0, which has been calculated as an approximate equivalent value to the Manning's n of 0.010, for the initial water depth of 0.3m at the upstream end of the model.

The final calculated depths of water produced in both ISIS and MIKE 11 are presented in Graphs 1 to 12, Appendix A.

4.2 Analysis of MIKE 11 Results – Applied Bed Friction, Manning's n = 0.010

It can be seen from Graphs 1 to 4 that the water level histories produced in MIKE 11 at the benchmarking stations S1 to S4 only very approximately follow the recorded experimental data.

The best calculated results are produced at station S1 where the general trend of the calculated history approximately follows that of the experimental data. Both experimental and calculated water levels start at 0.3m and quickly drop to an approximate level of 0.16m after 4.0s. After this time however, the calculated water level remains constant at approximately 0.165m while the experimental data shows a continued drop in water level until approximately 7.5s when the rate of change of water level slightly increases.

The calculated water level histories at the remaining three benchmarking stations S2, S3, and S4, show different behaviour in the change in water level to that of the experimental data.

It can be seen from Graph 2 that the experimental water level recorded at station S2 changes from 0.003m after 3.0s of the simulation. This change in water level corresponds to the arrival of the surge wavefront at station S2. Therefore, it can be determined that the surge wave has travelled 6.1m in 3.0s, giving rise to an approximate travelling velocity of 2.0m/s. It can then be seen from the plot of the experimental water level history that the water level only rises by a very small amount between 3.5s and 7.3s, after which time the water level increases rapidly, settling at an approximate level of 0.23m after 8.5s. The level remains approximately constant thereafter for the remainder of the simulation. This recorded rise in the water level after 7.3s is due to the superimposition of the resultant reflected wave caused by the angled walls at the front of the constriction on the travelling surge wave.

It can be seen from Graph 2 that the calculated water level history produced in MIKE 11 does not follow this same process of change as that observed experimentally. The water levels do not settle at any time during the simulation, but instead fluctuate broadly about the experimental levels. This suggests that the solution is not as stable as indicated by the animated profile plot produced within MIKE 11. However, an approximate increase in the water depth equivalent to that produced experimentally is produced by the software, demonstrating that the superimposition of the incident and reflected waves is calculated by the software. It must be noted that the final calculated depth of water in the channel at station S2 is only 0.02m higher than that recorded in the experimental data. The final calculated depths of water at stations S3 and S4 however are not so close to the experimental depths.

It can be seen from Graph 3, presenting the results at station S3, that the software calculated results fluctuate around the experimental data after 8.0s, which broadly indicates instabilities with the result. The final calculated water level produced by the software at station S4 is much greater than that produced in the laboratory test.

Despite the instabilities, the calculated results provide useful information regarding the speed of the propagating wave. It can be seen from Graphs 2, 3 and 4 that the calculated water levels show a delay in the arrival time of the initial surge wave. This is indicated by the time at which the water level changes from the starting level of 0.003m, the initial water depth downstream of the gate. The surge wave produced in the physical model can be observed to arrive at stations S2, S3, and S4 at 3.0s, 4.3s and 5.5s respectively. The same corresponding calculated arrival times can be observed as 4.2s in Graph 2, 6.5s in Graph 3, and 8.6s in Graph 4. This is an important finding of the software as the same delayed behaviour of the arrival time of the surge wave was observed in the calculated results of the two-dimensional models used to model the same dam-break in the original CADAM project.

4.3 Analysis of ISIS Results – Applied Bed Friction, Manning’s n = 0.010.

It can be seen from Graphs 5 to 8 that the results produced within ISIS are very unstable, as highlighted earlier. However, some approximately similar behaviour to the experimental data can be observed.

Firstly, it can be seen from Graph 5 that the general trend of the fall in the calculated water level at station S1 is approximately the same as the fall recorded in the laboratory test. The water level history produced by ISIS shows that ISIS does not calculate a constant water level at station S1 after 4.0s as calculated in MIKE 11. The ISIS water levels show a similar though slightly lower and fluctuating water surface profile when compared to the experimental results.

The calculated water levels produced at station S2 show very unstable results between the times of 1.0s and 4.5s. As such, it is impossible to draw any conclusive remarks from these results. However, it must be noted that the final calculated water level produced in ISIS at station S1 of 0.22m is only 0.01m different from the recorded experimental level.

The calculated water level history plotted in Graph 7 also shows significant instabilities between 1.0 second and 4.5s. However, the solution can be observed to steady significantly after 5.5s. As poor model convergence was only observed in ISIS for the first 3.6s of the simulation the solution will have settled significantly by 5.5s. As such, the rise in the calculated water level just before 4.5s in Graph 7 may indicate the arrival time of the surge wave at station S3. If this is the case, then ISIS will have approximately simulated the correct propagation velocity of the initial surge wave, but calculated a slightly higher continuous depth of water between 5.5s and 10.0s of 0.14m as opposed to the experimental depth of 0.11m.

The water level history produced in ISIS at station S4 shows very irregular water levels from approximately 3.5s until the end of the simulation. As these levels at no time become steady it is also impossible to draw any conclusions from these.

It should be noted that when undertaking the simulations with ISIS the default minimum water depth parameter of 0.01m was used. Alternative, smaller values were not investigated. However, inspection of the results shows that the depth of flow at time = 0.0s is less than this value (0.003m), as prescribed by the boundary data conditions. Furthermore, results at other times are also less than 0.01m. This anomaly has not been investigated further as part of the study.

4.4 Analysis of Applied Bed Friction with MIKE 11 (Chézy C = 70)

In a further attempt to improve the stability of the model in MIKE 11, the test was performed a second time, but using a Chézy C = 70, calculated as an approximate equivalent Chézy value to the Manning's n = 0.010, for the initial water depth of 0.3m upstream from the virtual sluice gate. It can be seen from Graphs 9 to 12 that applying the friction in this way to the model has no obvious effect on the results (in comparison to Graphs 1 to 4), with the calculated water levels being almost exactly equal to those generated with Manning's n = 0.010.

5 DISCUSSION AND CONCLUSIONS

The test is an extreme investigation of the capabilities of the software packages due to the nature of the hydraulic condition. As a result several notable nuances between the software packages have been highlighted.

HEC-RAS, which is capable of setting up the test, is unable to produce any results due the limitation of a 1 second time step (minimum) that can be used for a simulation. In many modelling scenarios running a model with a small time step can assist the modeller in developing good and stable initial conditions with which to undertake subsequent simulations with larger time steps. This inability of HEC-RAS may in due course limit or hinder the modeller in undertaking studies of a much simpler nature that may have complex/tricky initial conditions.

ISIS, which is capable of undertaking the simulations, has required the use of initial conditions from the partial wave development without which unstable results prevail. Conversely, MIKE 11 is capable of undertaking the simulations with either no wave or partial wave development. This may suggest that the numerical scheme adopted by MIKE 11 may be more robust than that employed by ISIS for conditions where there are significant/extreme velocity or water level changes.

Having to use fractions of an hour for the simulation time in ISIS can be problematic when attempting to select an appropriate time step and save interval. The ability of MIKE 11 to use hours, minutes or seconds, and fractions thereof, is a distinct benefit.

Only MIKE 11 was able to produce stable solutions for the dam-break progressive wave; however, the results calculated by this solution were not truly representative of the problem. This is likely to be due to the one-dimensional nature of the solution generated by the software. The test is not entirely appropriate for one-dimensional analysis as the water surface becomes very irregular when the surge wave hits the constriction.

The calculated water levels for MIKE 11 show a delay in the arrival time of the initial surge wave. This is an important finding of the software as the same delayed behaviour of the arrival time of the surge wave was observed in the calculated results of the two-dimensional models used to model the same dam-break in the original CADAM project.

The results from ISIS are inconclusive due to notable instabilities in the calculated water levels; however, the final calculated depth of water produced by ISIS at station S4 is exactly equal to that produced in MIKE 11.

In both ISIS and MIKE 11 the stability of the results is improved by increasing the implicit weighting factor in the numerical scheme. Within ISIS, which uses the Preissmann 4-point implicit scheme, a value of 1.0 has been required, whereas in MIKE 11, which uses Abbott 6-point implicit scheme, a value of 0.65 has been sufficient. Again this may suggest that that the numerical scheme adopted by MIKE 11 may be more robust than that employed by ISIS for conditions where there are significant/extreme velocity or water level changes.

It is anticipated that improved results could be achieved from each of the software packages through refinement of the calculation settings and options. For instance, if the “Delh” value (the depth at which the slot is activated) is reduced or an alternative to the phasing of the convective term is used (i.e. by setting FroudeMax and FroudeExp to 1 and 2 respectively in the HD parameter file) then improved results could be anticipated.

6 RECOMMENDATIONS

Although this test has been an extreme investigation of the capabilities of the software packages there are several notable areas where they may benefit from additional functionality, such as:

- Within HEC-RAS the user can only select a time step and the save interval from a predefined list of incremental values. The smallest increment for the time step is 1 second, which for this test has been a significant disadvantage. The ability to undertake a simulation with a time step and save interval of the modeller's choice would in the opinion of the testers provide much needed flexibility and functionality, especially during model development.

- Within ISIS the user needs to specify the run time in hours and fractions thereof. The ability to have the option to choose hours, minutes or seconds as the units for specification would, in the opinion of the testers, provide greater flexibility for the modeller.

It is anticipated that improved results could be achieved from the software packages through refinement of the calculation settings and options. Hence, it is recommended that any future testing should attempt to find the most appropriate settings for this test and tests of this nature.

The modelling approach required for this test has not provided a true real life test of the software packages, hence it is recommended that the test specification be enhanced and added to so as to consider a more real life prototype situation.

Some of the software packages have the functionality to model dam-break and levee breach, which have not been tested within the current scope of the test specification. Hence, it is recommended that the test specification be refined so as to reflect real life conditions. This may merit the undertaking of further experimental studies so as to provide appropriate validation data.

7 REFERENCES

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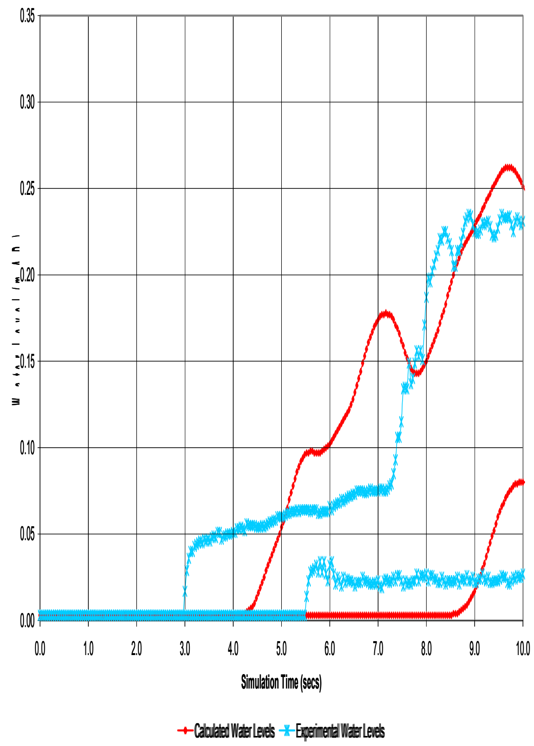
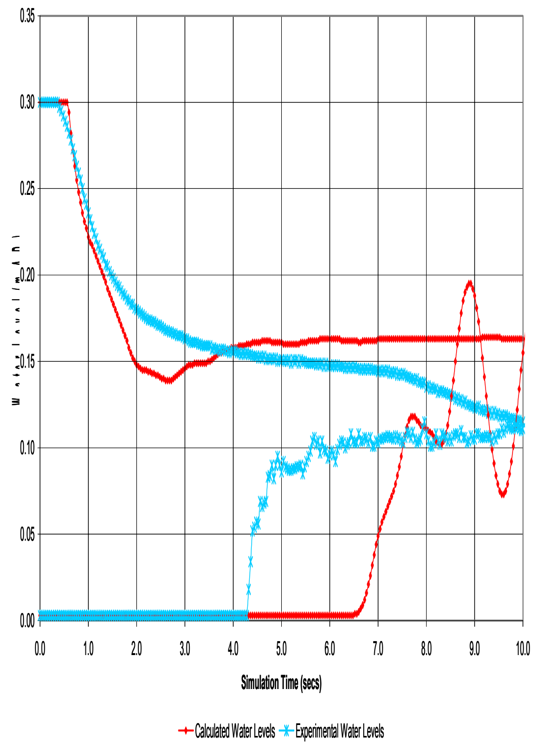
APPENDIX A RESULTS

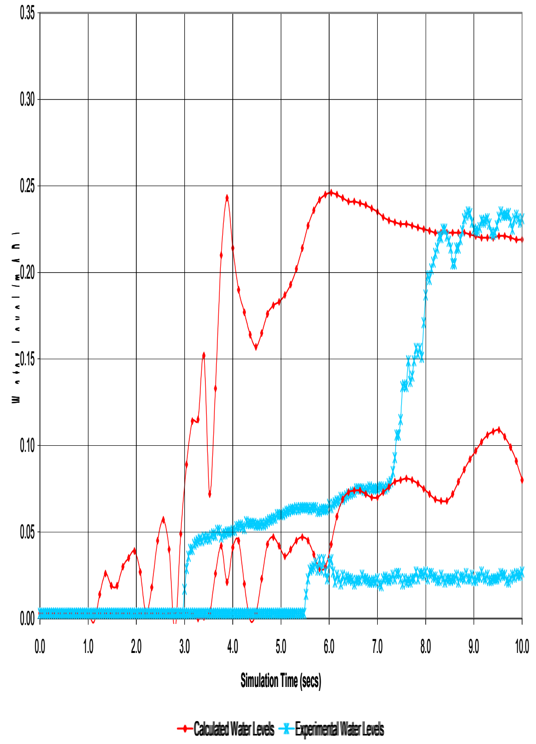
Graph 2 - MIKE11 - Calculated and Experimental Water Level Histories at Location S2
Applied Bed Friction - Manning's n=0.01



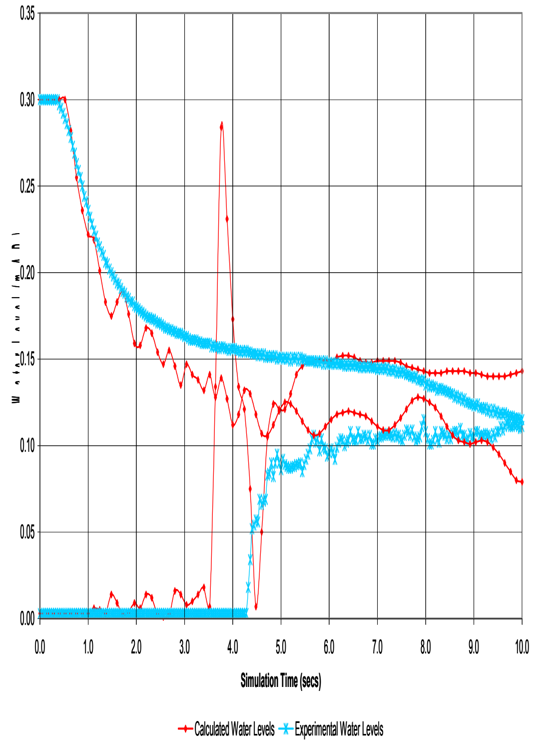
Graph 3 - MIKE11 - Calculated and Experimental Water Level Histories at Location S3
Applied Bed Friction - Manning's n=0.01



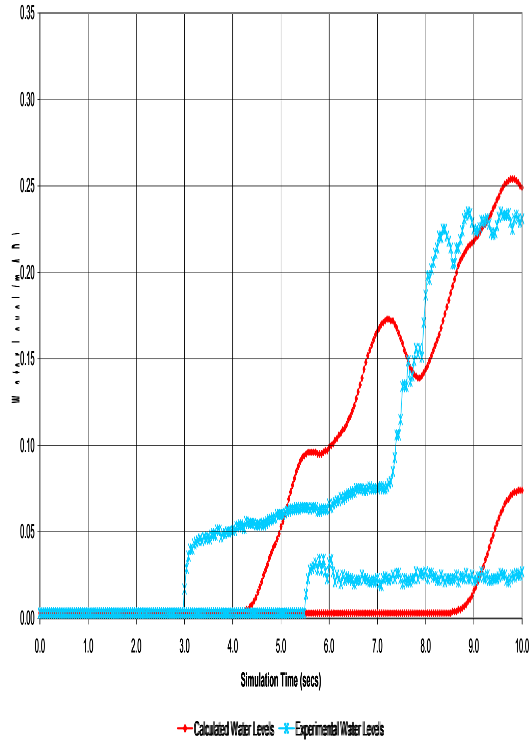
Graph 6 - ISIS - Calculated and Experimental Water Level Histories at Location S2
Applied Bed Friction - Manning's n=0.01



Graph 7 - ISIS - Calculated and Experimental Water Level Histories at Location S3
Applied Bed Friction - Manning's n=0.01



Graph 10 - MIKE11 - Calculated and Experimental Water Level Histories at Location S2
Applied Bed Friction - Chézy C=70



Graph 9 - MIKE11 - Calculated and Experimental Water Level Histories at Location S3
Applied Bed Friction - Chézy C=70